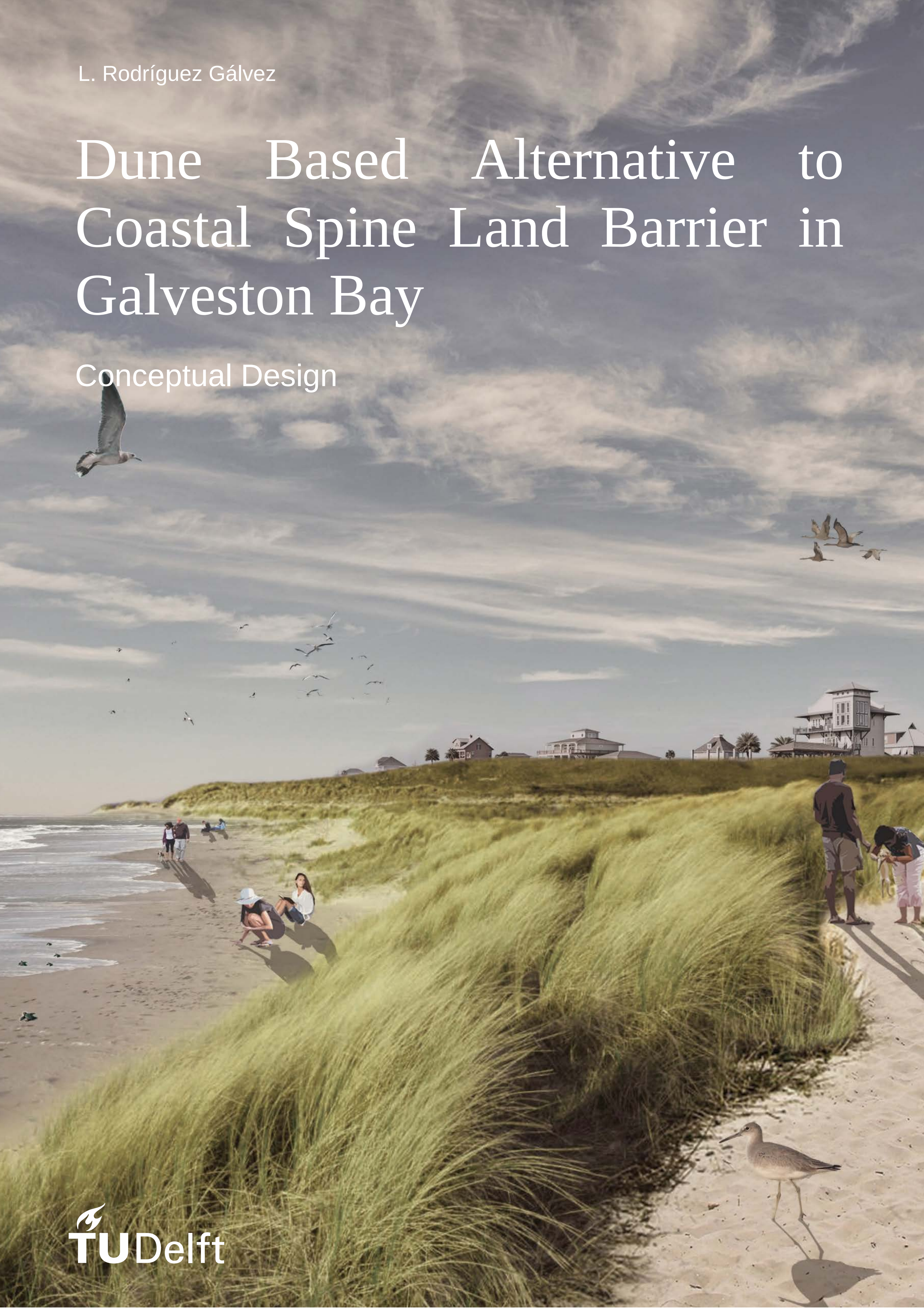
L. Rodríguez Gálvez

Dune Based Alternative to Coastal Spine Land Barrier in Galveston Bay

Conceptual Design



TU Delft

Dune Based Alternative to Coastal Spine Land Barrier in Galveston Bay

Conceptual Design

by

Luis Rodríguez Gálvez

in partial fulfilment of the requirements for the degree of

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Cover image: Conceptual design illustration of the dune system along Galveston Bay coast.

Delft University of Technology

Preface

This thesis represents the conclusion to my master studies in Hydraulic Engineering at TU Delft. It covers the initial design, evaluation and assessment of a dune based alternative to the land barrier proposal in the Coastal Spine plan for Galveston Bay, Texas.

I would like to express my gratitude to everyone that made the elaboration of this thesis possible, starting from my committee members. Sierd, thanks for your guidance, advice, patience and support throughout the whole process, always pointing to the right direction and giving a positive focus to my thoughts every time I needed it. Thanks Jeremy for helping during critical parts where your orientation helped me moving forward. Bas, thanks for your feedback and broad perspective of the topic, your expertise in the area gave room to countless improvements. Additionally, I would like to thanks Baukje for her enthusiasm and interest, finding new perspectives for my work. Lastly, thanks to Jos and Erik for their assist with their knowledge in the topic.

Thanks to the friends I made during my master life, with who I had a great time. Especially to Stephen, Luisa and Juan, this time would have not been the same without the countless moments of joy we had together. Thanks to the fellow master students of room 3.92, working next to you this period made it more manageable and it has been a pleasure getting to know you. And of course, thanks to Maria for being there with me at all times, sharing good moments and always with kind and encouraging words helping me to overcome difficult ones. Lastly I want to thank my parents and family, always supporting me unconditionally in every decision I make, I hope I can make you proud.

L. Rodríguez Gálvez
Delft, August 2019

Abstract

Galveston Bay is located in the Gulf of Mexico, in the Upper Texas Coast. The Port of Houston, one of the most important ones in the country, makes it a buoyant economic centre of Texas and U.S, being accessed through the bay via the Houston Shipping Channel. In terms of environment, it serves as a home for different species of birds and waterbirds, dolphins and sea turtles, among others. Wetlands are found in shallow areas of the bay, although they are losing presence over time. Another problem Galveston Bay is facing is beach retreat, making imperative the constant nourishment of different coast stretches to preserve their value.

Historically and in recent history Galveston Bay has been struck by a large number of severe hurricanes. It is listed as one of the most hurricane prone areas in the world. The impact of hurricane Ike in 2008 raised awareness of this issue, leading to different studies on how to tackle the situation increasing Galveston Bay flood protection. The project that has been getting more strength is the Coastal Spine or Ike Dike. It consists on a series of connected dikes and storm surge barriers that close off the bay from the gulf during an event. In this thesis it is proposed a dune system as an alternative to the dikes as land barrier in order to provide a more spatial integral solution in line with the area characteristics. The concept consists of using a dune system that can serve as a flood defence structure in case of hurricane while being integrated into the coastal system in daily conditions

With the hurricane 1/100 yr^{-1} boundary conditions as design values, the initial dune design is characterized by a crest level of MSL +7.5 m, a width at MSL of 100 m, slopes ratio of 1:3 and dune crest width of 55 m. Having the dune initial parameters, modelling with XBeach is used to evaluate the behaviour of the dune in design storm dynamic conditions, which are obtained by taking hurricane Ike as a reference. After several iterative model runs it is determined that the initially estimated dune has the best relative performance. A dune erosion 187.5 m^3/m takes place along the design hurricane duration, with 40% of the crest remaining after the storm. The effect of vegetation on the dune model is takes place mainly at the dune foot. The possibility of a two level dune is explored, although results are not promising.

Dune design a construction costs amount to \$3.06 billion is, which compared to the Coastal Spine dike construction costs of \$2.25 billion, is relatively more expensive. However, the maintenance cost analysis, comprising post storm recovery, erosion due to lesser storms and sea level rise dune upgrade, provides a simple straightforward estimation of most of the expenses the dune has during its lifetime, while this analysis has not been done for the Coastal Spine dike solution and is expected to surpass the dune long term costs, thus balancing the combined construction and maintenance costs across both alternatives.

A qualitative comparison between the dune and dike designs different features regarding the scope of the project as well as its impact within the area slightly favours the implementation of a backshore dune. In order to take advantage of the philosophy behind the dune design, a two design alternative concept is presented. The first design focuses mainly in the flood protection aspect, while the second one promotes the natural value of the area including a second dune line seawards creating a dynamic intertidal area in between.

Preliminary results yield positive findings regarding the use of the design dune as coastal flood defence structure given the design hurricane boundary conditions. In terms of costs, an estimation of the dike maintenance should be obtained to provide a conclusive argument supporting one alternative over the other. Morphodynamic assessment of a dune design partially interfering with the shoreline can lead to potential dune design optimisation.

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1. Introduction

1.1. Galveston Bay Description

Galveston Bay is an estuary located in the Gulf of Mexico coast of the United States, in the Upper Texas Coast of the state of Texas. Remarkable urban areas surrounding the bay are Texas City, Baytown, Pasadena and, further inland into the north-west, Houston, the most populated city in Texas and fourth in the whole United States with 2.3 million inhabitants ("U.S. Census Bureau," 2017). The configuration of the bay can be seen in Figure 1.

The Port of Houston can be considered one of the main assets of the area in terms of economy as it contributes with more than 1.1 million and 3.2 million jobs in Texas and globally respectively, having an estimated economic value of more than $800 billion (Port of Houston Authority & Martin Associates, 2014). In terms of overall tonnage, it is ranked the second one in the U.S. (Port of Houston Authority, 2018). Galveston Bay is key within Texas fishery industry, with different types of shrimp, crabs and oysters among others being the major part of the fishing industry resources in the area (Larry, Spear, Taylor, & Thatcher, 2017). Another source of income in the area is the petrochemical refining industry, especially in Texas City (Marathon Petroleum, 2018).

The bay water surface area accounts for around 1.550 square kilometres (US EPA, 2005), making it the seventh largest estuary in the US and the second in the Gulf of Mexico (Salas-Monreal, Anis, & Salas-de-Leon, 2018). It is inland fed by Trinity River and San Jacinto River and it comprises four main sub-bays: Galveston Bay (core) Trinity Bay, East Bay and West Bay (Figure 1).

Figure 1. Galveston Bay area land features and cities. Source: (Texas State Historical Association, 2010)

The bay is constituted by a semi-enclosed water body divided from the Atlantic Ocean by Bolivar Peninsula (north) and Galveston Island (south). In a hurricane scenario, they act as a natural line of defence against flooding in the surroundings of the bay from ocean storm surge and flow. There are three inlets which let the water exchange take place between bay and the gulf: Bolivar Roads inlet, San Luis Pass and Rollover Pass (Figure 1). The first one is located between Bolivar Peninsula and Galveston Island and is the major agent when it comes to water exchange, accounting for more than 80 percent of the total between bay and ocean. The Houston Ship Channel goes through this inlet and crosses the bay until the Port of Houston. San Luis Pass is located at the western end of Galveston Island and is responsible for less than 20 percent of the tidal exchange. The remaining, almost negligible, water exchange takes place last one is Rollover (Fish) Pass, a man made inlet in the narrowest part of the Bolivar Peninsula, connecting it to the main land. It was originally made to improve water quality and allow for fish migration in and out of the bay (Galveston Bay Estuary Program, 2011).

In terms of ecology, the bay is the ecosystem of a variety of wildlife species, such as colonial waterbirds, shorebirds, dolphins and sea turtles, among others (McFarlane, Newell, Rifai, Bedient, & Shipley, 1994). Shellfish industry is relevant, as shrimp, crabs and oysters are found in this habitat and harvested by locals.

Apart from being of significant economically importance, its ecosystem can be seen as a health indicator of the bay (Houston Advanced Research Center & Galveston Bay Foundation, 2018).

The Upper Texas Coast is one of the most hurricane prone areas in the world (Roth, 2010). A map showing the path of the different tropical storms and hurricanes taking place since 1851 with their respective categories can be seen in Figure 2, accounting for a total of 64 hurricanes and 56 tropical storms.

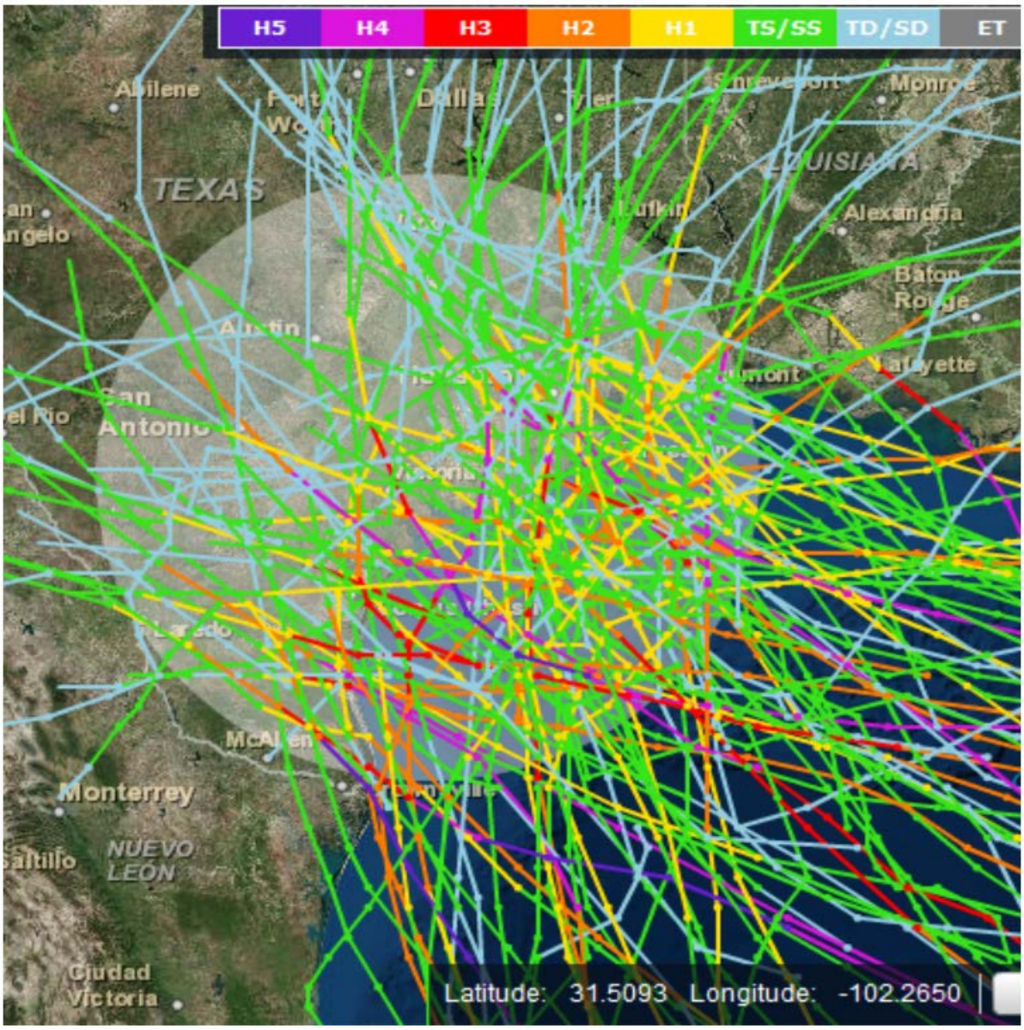

Figure 2. Tropical storm and hurricane paths in Upper Texas Coast since 1851. Source: (Pannell, 2015)

The top hurricanes and tropical storms that have struck Upper Texas Coast in the latest times, based on estimated economic loss from damages are presented in Table 1, also showing the present value of the damages obtained by applying the inflation rate since the date of the event. According to recent studies, the landfall of tropical storms and hurricanes have experienced an increase in the past century, to the point that a tropical storm strikes the Texas coast three times every four years (Roth, 2010). In other terms, Upper Texas Coast is the landfall location of a major hurricane (category 3 or more) on average every six years (Anderson, 2007).

It is worth mentioning that the Galveston Hurricane of 1900 (Great Storm) (Figure 3), even though it is at the bottom of Table 1 list, it had the biggest impact in Galveston history. It is considered the U.S. worst and deadliest natural disaster in history, with an estimated number of casualties ranged between 8,000 and 12,000. It definitely changed Galveston City history, which lost its never recovered relevance as port and regional economic centre. As an order of magnitude, the highest Galveston City point was at MSL +2.7 m an the hurricane surge is estimated to reach a level of MSL +4.6 m (Cline, 2004).

Storm	Date	Landfall	Damage costs	Current value
Hurricane Ike	13/09/2008	Galveston	$29.5 billion	$35 billion
Tropical Storm Allison	05/06/2001	Freeport	$5 billion	$7.2 billion
Hurricane Alicia	18/08/1983	Galveston	$1.8 billion	$4.6 billion
Hurricane Dolly	23/07/2008	South Padre Island	$1 billion	$1.2 billion
Tropical Storm Allison	26/06/1989	Freeport	$500 million	$1 billion
Tropical Storm Frances	19/09/1998	Corpus Christi	$500 million	$785 million
Hurricane Celia	03/08/1970	Corpus Christi	$400 million	$2.6 billion
Tropical Storm Claudette	24/07/1979	Texas-Louisiana border	$400 million	$1.4 billion
Hurricane Carla	11/09/1961	Port Lavaca	$400 million	$3.4 billion
Hurricane Allen	09/08/1980	Port Mansfield	$300 million	$932 million
1900 Galveston Hurricane	08/09/1900	Galveston	$35.4 million	$1 billion

Table 1. Ranked Upper Texas Coast tropical storms and hurricanes

Figure 3. Galveston after the 1900 Hurricane. Source: Texas State Library

Looking at the closest events in time, it is reasonable to conclude that the coast is very vulnerable to storm events, which has been proven by Hurricane Ike in 2008 and, more recently, by Hurricane Harvey in 2017. The former event caused more than $30 billion in damage and dozens of deaths (Blackburn & Bedient, 2010). Its impact (Figure 4), along with the aforementioned tropical storm history, have been the trigger for the development of projects focused on the strengthening and improvement of the Coastal Spine (Ike Dike). These projects are based on the idea of limiting the inflow into the bay area to reduce the surge mainly by means of barriers and seawalls, based on the delta design approach followed in The Netherlands (van Berchum et al., 2015).

Furthermore, the flood defence system has been a topic for several master thesis, such as the probabilistic design of the Land Barrier (Plomp, 2016) or the hurricane surge risk reduction strategy (Stoeten, 2013). Different nature based solutions have also been studied, such as oyster reefs and wetlands (de Boer, 2015) or the effect of wave attenuation due to vegetation (Godfroij, 2017). Other hybrid solutions have been under research, such as placing sandy sediment at the foot of the existing seawall (Muller, 2015).

Figure 4. Hurricane Ike flooding consequences in Galveston Island, September 13th 2008. Source: https://www.texastribune.org

However, there has not been an in depth assessment of alternative coastal flooding solutions applicable in the interface between estuary and ocean that do not rely on hard structures. All of the aforementioned studies are either located inside the bay or include hard structures in their concept of providing coastal flooding defence.

1.2. Problem

Galveston Bay region has experienced a rapid economic during mid-20th century; petrochemical and oil industry, the development of the Port of Houston, construction of highways and growth of the communities' urbanization (Texas State Historical Association, 2009) have been patterns that help to understand the current state of the area. This fast development, carries an increase in vulnerability of the area since the damages in case of a specific flooding episode grow with the flood exposure and the possible damages. This is even more devastating with urban development, being more costly and difficult to face (Jha, Bloch, & Lamond, 2011). All of it, combined with the effects of sea level rise and increased hurricane occurrence due to global warming (GFDL & NOAA, 2018).

All of the previous arguments call for an effective strategy to prevent and fight floods. Historical and recent event data suggest that an improvement in the flood defence system could potentially prevent economic losses, save lives and protect the ecosystem. The current strategy against flooding is passive rather than proactive, the effects of an event are mitigated once the damage has been done. Nourishments are carried at specific locations until it is washed out and then, a new nourishment is placed; it can be addressed as a patching strategy (Anderson, 2007).

After Hurricane Ike in 2008, awareness of the flooding issue in the area has arisen again and there is a trend in shifting flood defence strategies into a more regional coordinated approach. One system design, which has gained a lot of strength in the recent past years is the Coastal Spine or Ike Dike. The proposed design, following the Delta Design, consists of a combination of dikes, seawalls and barriers (Figure 5). This design has been determined as the preferred choice for protecting Galveston Bay from future storm surges (Powell, 2018), taking part in its development, among others, Texas A&M University or the U.S. Army Corps of Engineers.

Figure 5. Preliminary Coastal Spine design concept. Source: (Kothuis et al., 2015)

As it has been explained in the previous section, alternative studies and solutions have been provided, analysed and studied in different MSc thesis, for options in both within the bay and barrier islands. In general terms, the ones that focused inside the bay, proposed a Building with Nature alternative assessment or evaluation; while the topics focused in the barrier islands followed a risk assessment based on probabilistic approaches of different Ike Dike options and hard structures. However, alternatives that do not rely on hard structures have not been designed, assessed or compared to the Coastal Spine land barrier dike proposal. It would be an interesting option to consider an entirely soft intervention for the Coastal Spine land barrier component.

One of the main activities from which the area is gaining popularity is their tourism offer, which over the years has been overcoming the economic crisis started in 2009 (Figure 6). Galveston Bay offers a diversity of activities for visitors, being remarkable the ones related to its ecosystem. With over 600 bird species in the area, birdwatching has raised in popularity as an appealing activity (Galveston Bay Estuary Program, 2011). Galveston Island is heavily sustained by tourism revenue to the point that, in 2015, one third of the jobs are directly or indirectly related to it, generating an income of $279 million in that year (Tourism Economics, 2015). Galveston Island and Bolivar Peninsula beaches play an important role not only in the aforementioned leisure purposes and its effect in the ecosystem, but also when it comes to flood protection.

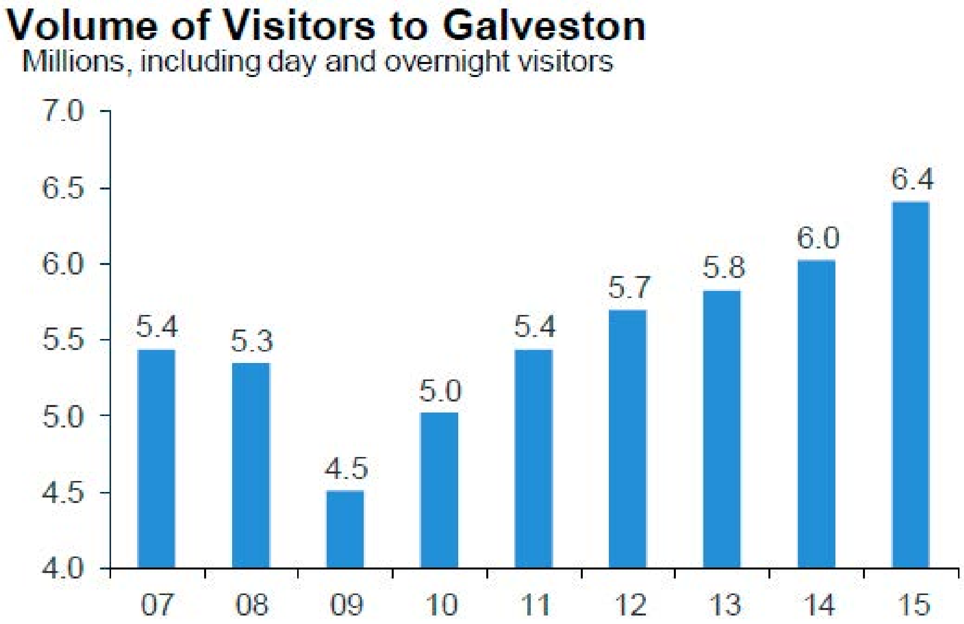

Figure 6. Galveston Bay tourism growth over the years. Source: (Tourism Economics, 2015)

Beach in the barrier islands is retreating on average from 1 to 1.5 m/yr, although this value considerably varies spatially, reaching values up to 4 - 4.5 m/yr (Paine, Caudle, & Andrews, 2014). Although technically the beach is retreating instead of disappearing, it poses a great threat to the coast. Residential beach houses reach the first line of the beach, but with its recession, the beach parcel in front of them is gradually reduced until it disappears. In final stance, these houses are at a stake since the shoreline reaches their location. In order to prevent it, local residents implemented their own private solutions, such as installing bulkheads in front of their property as an attempt to stop the retreat (Figure 7). As it can be seen, the retreat stopped by means of the hard structure installation, but at the cost of supressing the remaining beach in front of it.

Figure 7. Bulkheads in front of private properties in Galveston Island, west of the Seawall. Source: (Anderson, 2007)

Another erosion phenomenon due to the presence of a hard barrier altering sediment transport has been taking place since the Galveston Seawall was constructed in 1904. The current along the coast is west directed, thus erosion in the beach immediately west of the seawall is to be expected, as it can be observed in Figure 8.

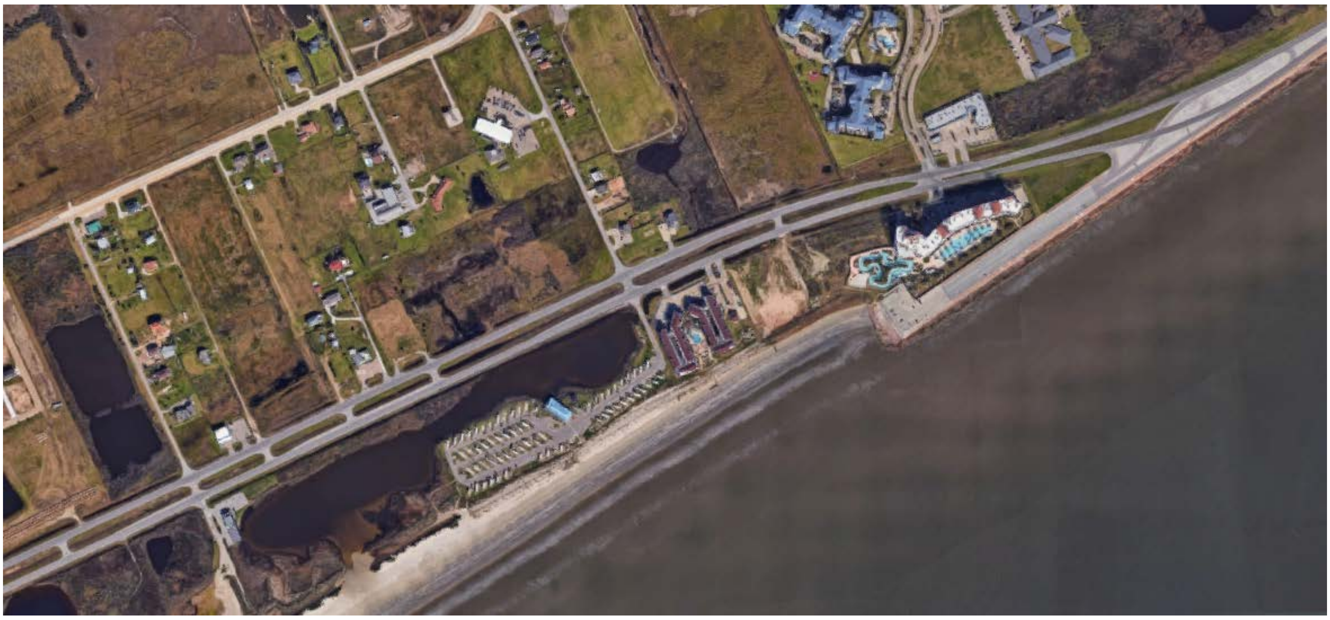

Figure 8. Beach erosion close to the west end of Galveston Seawall.

It can be seen in Figure 8 that the beach part of the coast in front of the seawall disappeared. The coast kept on retreating back, but the wall didn't allow for it until it was completely washed out. However, once the effect of the hard structure disappeared, it is seen that the beach has a more inland beginning that in the wall part of the shore cannot reach. The seawall acts as a barrier for floods and at the same time as a limit for beach development and processes. In Figure 9 it is displayed the shoreline envelope (range of shoreline positions) from 1930’s until 2012 in the same area as in Figure 8. It can be appreciated how the envelope, and thus the recession of the beach, is wider right after the seawall end, showing graphically its limiting effect.

Beach retreat is a natural phenomenon that is taking place all along the Bolivar Peninsula and Galveston Island at a steady rate that cannot be neglected when considering coastal interventions. Not taking it into account can lead ultimately to the vanishing of the beach, with all its related social, environmental and economic consequences. On the other hand, not acting puts at stake beachfront properties.

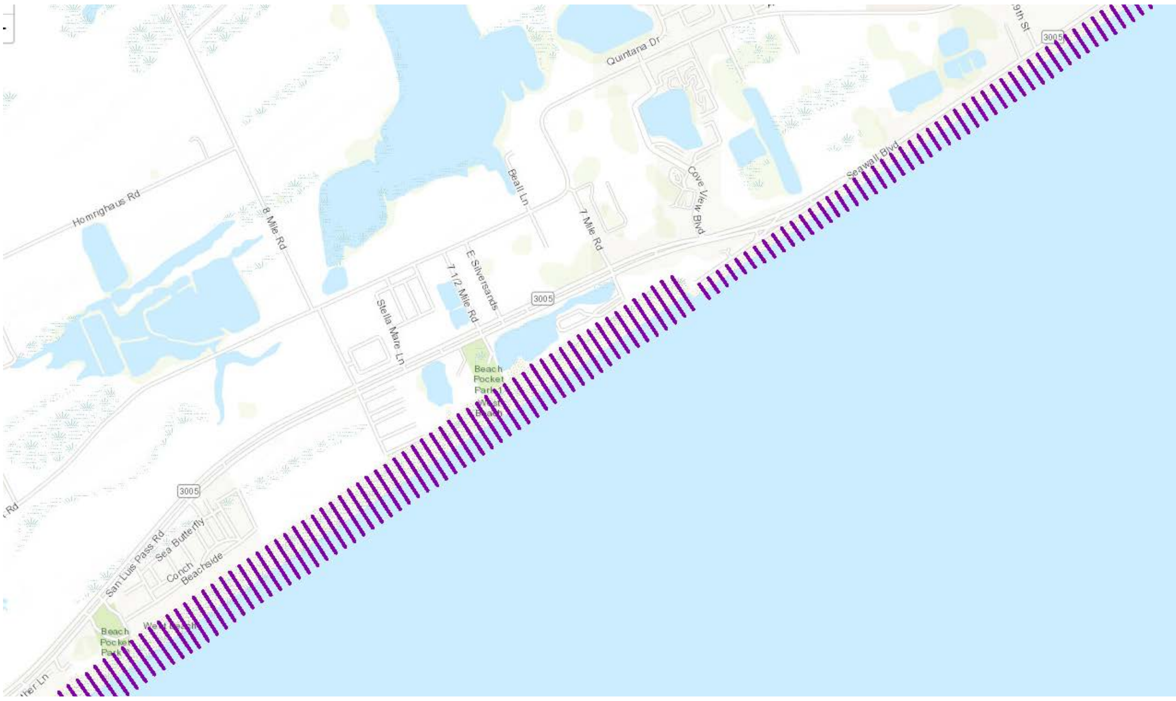

Figure 9. Coastline envelope from 1930’s until 2012. Source: (https://coastal.beg.utexas.edu/shorelinechange2012/)

The current situation is mainly a result of the different stakeholders and parties involved in coast management and flood protection interests, leading to a relatively slow, uncoordinated and non-regional reaction and planning to the threat that coastal flooding poses.

1.3. Objective

The ultimate goal of this study is to design a dune system that can serve as a land barrier in the Coastal Spine scheme, evaluate its performance under design hurricane conditions and reach a conclusion about its feasibility, comparing it with the current Coastal Spine land barrier proposal presented in *E.C. van Berchum et al., 2016*. In order to do so, a two-step solution will be considered. An initial design of a dune system that covers the entire Galveston Island and Bolivar Peninsula coast length will be proposed as a mean to relieve flooding impact in the area. Beach nourishment to keep it in a healthy state is of great relevance since if the retreat process goes on enough inland, it endangers the dune stability putting at stake the flood defence plan in case of an event scenario This way the short term (hurricane condition) as well as the long term (daily flow erosion) scenarios are addressed. Finally, the obtained design will be evaluated via numerical modelling (XBeach) and later compared with the most recent concept of Coastal Spine land barrier dike (van Berchum, de Vries, & de Kort, 2016), from an economical point of view as well as in terms of design bases and choices.

The nourishment maintenance is a key factor to the whole project. It is the only available tool to stop or control the retreat of the coast (Anderson, 2007). The coast tries to reach an equilibrium state in its profile with respect to water level (Figure 10). With a situation of relative sea level rise like the one being experienced in Upper Texas Coast (Gordon, 2013), this equilibrium is not reached and it becomes a dynamic coastal profile (Dean, 2005), thus always eroding the swash zone. If a dune system were to be placed in the back of the beach, not only it gets eroded by episodic events, but also there might be the case that the recess of the foreshore eventually reaches the dune system, endangering it. However, certain design choices explored in later chapters regarding dune allocation can minimize or completely prevent the erosion of the dune front as far as it is out of reach from the erosion influence zone.

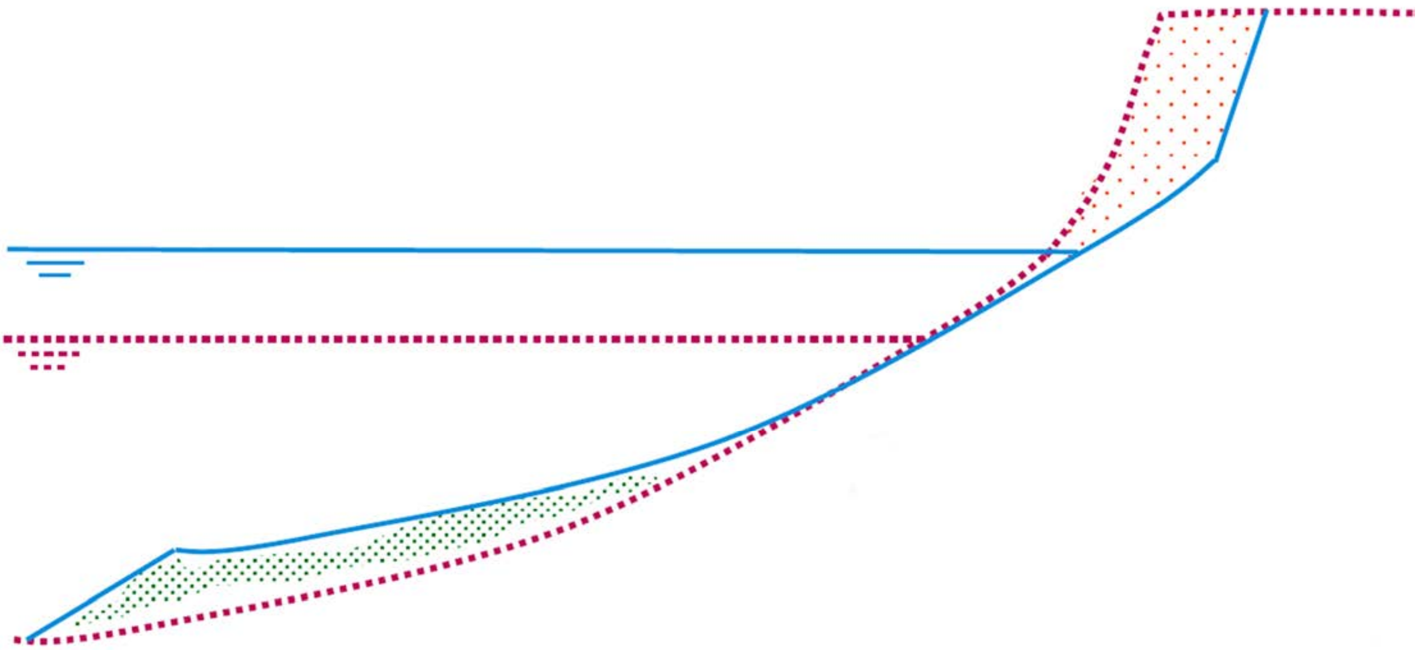

Figure 10. Equilibrium shoreface profile for two different water levels.

The proposed alternative should provide enough flood resilience, focusing and taking advantage of the natural strengths and processes. After a hurricane event, most of the sandy sediment remains within the coastal system and even, given enough time, eventually finds back its place in the equilibrium profile (Dean, 2005). This feature will be explored in further detail with help of numerical tools as it determines the dune needed degree of re-construction in a post-storm scenario.

The main research question of the thesis is as follows:

Is it feasible to implement a coastal flood risk reduction plan for Galveston Bay with a dune system instead of a hard structure as land barrier?

Subsequent follow-up subquestions are:

- How would a first system design and maintenance plan would be? Which requirements would it have and which assumptions would be needed to be formulated?
- How would the defence system react to the forcing the design hurricane scenario?
- How does it compare with the Coastal Spine hard structure option presented in *E.C. van Berchum et al., 2016*? What are the benefits and drawbacks of the two perspectives?

1.4. Research Method and Data Analysis

The boundaries of Galveston Bay Area are depicted in Figure 11, as well as the location of its important cities. It can be seen that it is comprised of a large extension of territory, thus designing and planning a whole defence system attending to the specific requirements (structural, geotechnical, stakeholders, landscape…) of every part of the whole Galveston-Bolivar stretch it is a task that would require an extremely high level of detail in every aspect of the process.

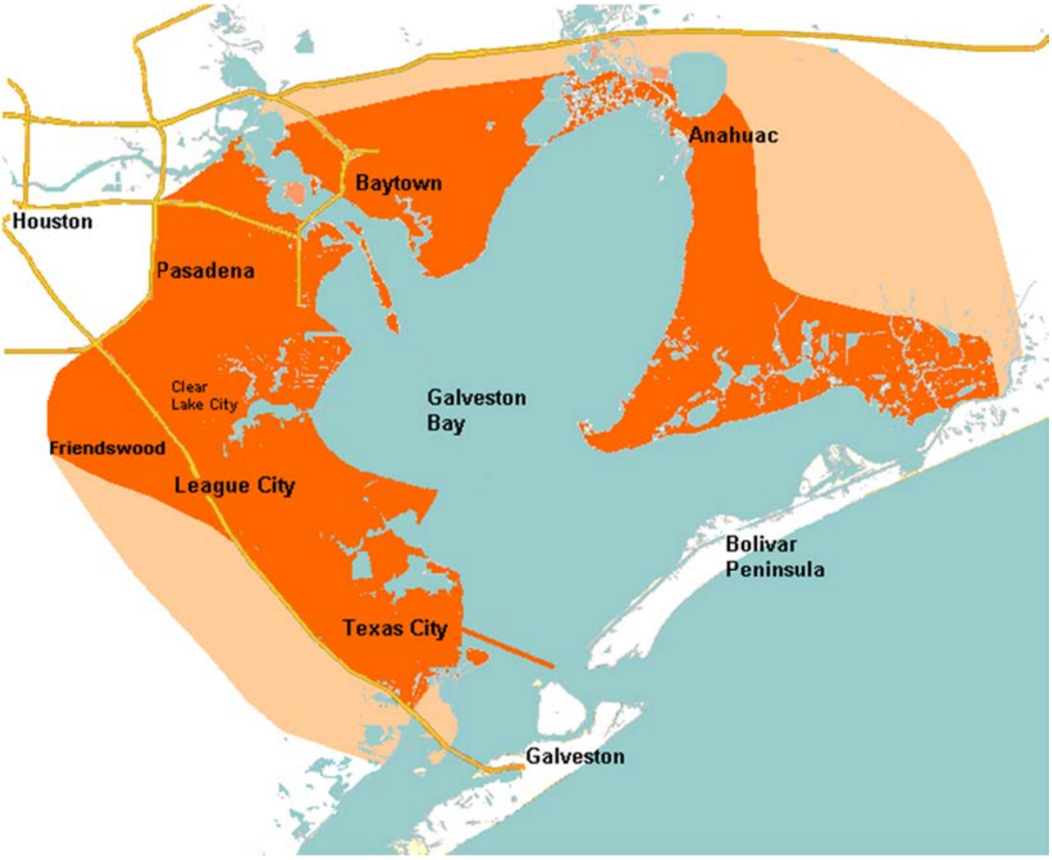

Figure 11. Galveston Bay estuary area, enclosed from the Gulf of Mexico by Galveston Island and Bolivar Peninsula. Source: (Galveston Bay Estuary Program, 2009)

In order to simplify the procedure to be able to obtain acceptable results within reasonable time frame, at first instance, the focus will be given to a particular cross section within the islands span. This cross sections will be selected based on two different criteria: morphology and land type. With this approach the spot which has more design limitations is given more attention and the design will be made according to the conditions of this place, making it applicable to the rest of the coast.

The islands will also be divided into different stretches with similar erosion/accretion rate and a representative section of each one will be obtained. An analysis of the beach profile can be performed and the dune will be planned to be located in its natural topography spot behind the backshore (Figure 12) when possible, minimizing visual impact and enhancing environmental value as well as increasing their distance to the erosion influence zone. From these profiles the sand volume needed for the construction phase will be obtained. As this project purpose is to provide an initial insight of a sandy solution against coastal flooding, the remaining of the dune design along the coast, will be assumed to be a smooth transition between them. It should not pose any problem since the crest of the dune is planned to be the same across the whole islands.

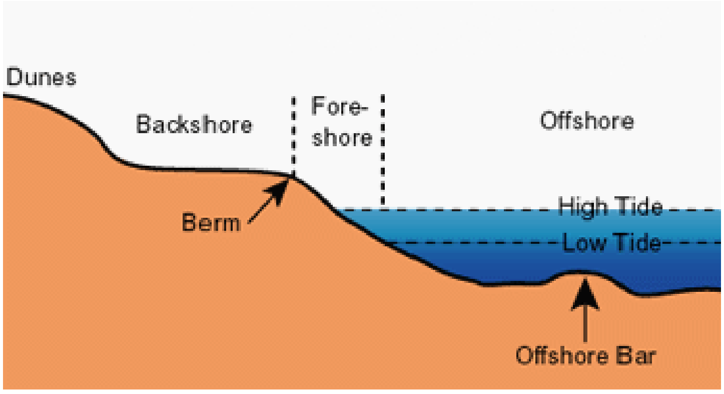

Figure 12. Left: Beach profile. Right: Dune example in California. Source: (https://www.californiabeaches.com)

Dune design with the specific purpose of providing coastal flood protection is an area not developed with high amount of depth, yet in this case holds potential for providing a reliable and resilient flood defence system that can blend in with the local environment, characterized by the presence of multiple national parks along the coast stretch of Galveston Bay with its fauna and flora. This thesis will take concepts of dike design such as overtopping criteria and combine them with others relative to sandy features like erosion in order to provide a first simple yet logical guideline that accounts for the main parameters for flood protection dune design.

In order to assess the effectivity of the dune system, modelling will be used. To that purpose, XBeach is the selected tool to perform the computations since it is able to capture short term morphological changes associated to hydrodynamic storm forces reliably in field studies and laboratory tests (Berard, Mulligan, Ferreira da Silva, & Dibajnia, 2017) (Williams, Esteves, & Rochford, 2015) (Terlouw, 2013) . The process of evaluating the dune system flood prevention is next explained stepwise, with its assumptions, benefits and drawbacks.

Firstly, an early design of the dune must be obtained for its later performance evaluation in an event situation. The main parameters in this case are dune height and dune width, which can be obtained from an overtopping and post storm dune recession analysis respectively. Unlike the case of a conventional dike, dunes fulfilling flood protection duty are not advised to be overflown since the areas most eroded during an event are the closest ones with respect to the maximum run up level (Łabuz, 2014).

With the initial dune design based on the mentioned parameters among others (slope, height, width…), a two design alternative differentiated in the local natural value enhancement will be formulated in the later discussion. As it will be explained in Chapter 2, the site characteristics of the islands stretch is not homogeneous along the coast. This means that there are some parts that are scarcely populated while others like Galveston are urban areas. Others, like the western end of Galveston Island is bordering, national park territory, in this case Brazoria National Wildlife Refuge (U.S. Fish & Wildlife Service, 2018) (Figure 13).

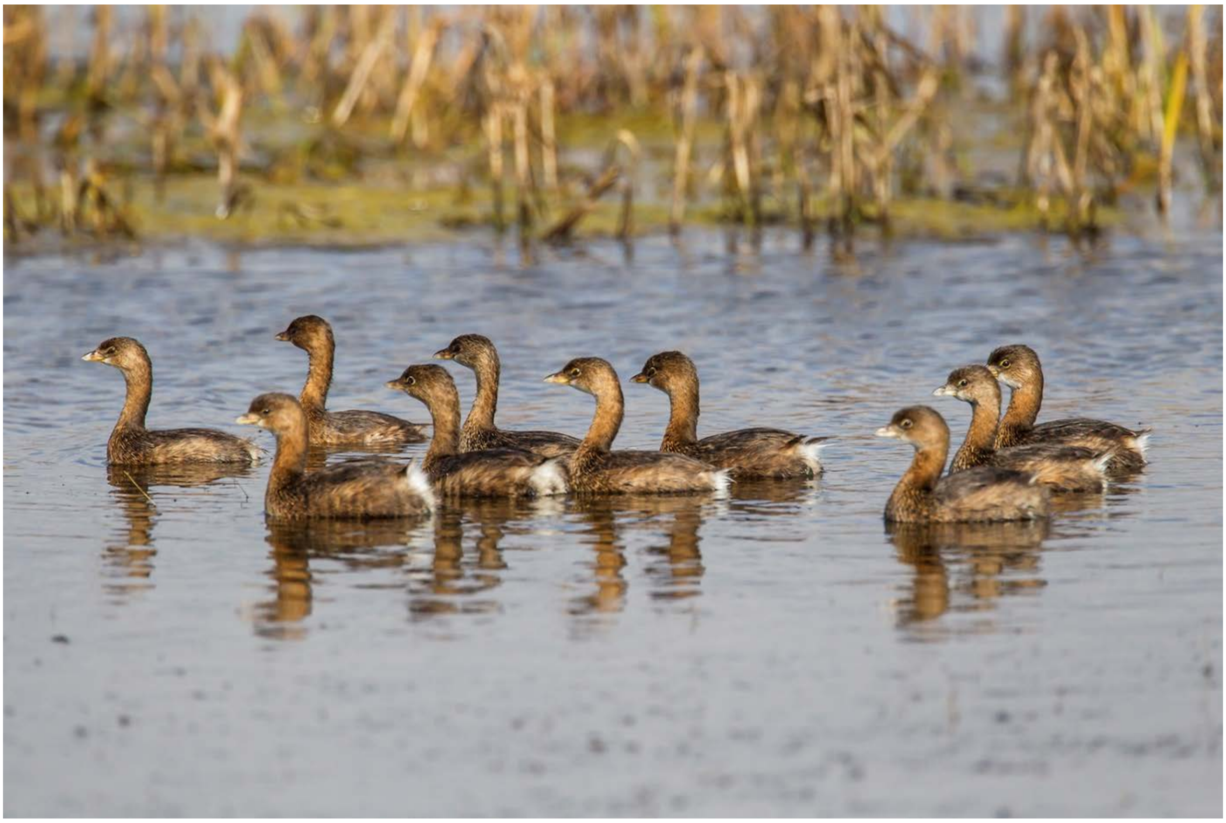

Figure 13. Pied-billed grebe water birds at Brazoria National Wildlife Refuge. Source: (Clark, 2016)

The two design concept comes from the idea that due to the disparity of the land and its use between zones, there is no design that fits all. It is considered then that one of the two designs has focus in its nature enhancing value while the other is a more practical solution. Nevertheless, both are designed to meet the same flood protection standard.

As stated before, numerical modelling, Xbeach specifically will be used to further refine the design. Visualizing the design in a storm situation will be possible, what will induce another tool to improve and fix problems with the design. Xbeach is chosen because due to its coastal nature it is able to model sand morphological changes when interacting with different hydraulic conditions.

As stated before, if the intention is to keep having a beach zone at the coast nourishment is a necessary measure since retreat will keep on being an ongoing process. With this intervention, the recession can still take place at the back side of the protection, unlike with a hard structure like a seawall which acts as a barrier to the movement of the shoreline. This makes the soft solution a more flexible design.

The last part involves the comparison between the dune design and the one presented at *E.C. van Berchum et al., 2016*. Costs, including the post storm recovery scenario in the case of the dune system along with maintenance and sea level rise costs not analysed in previous Ike Dike reports, will be compared. In order to have a global perspective, in the final discussion different criteria will be analysed and compared between the designs such as vulnerability, construction or natural value, The most notable differences will be marked and finally, a conclusion will be drawn out of the design process and results yielded.

1.5. Thesis Outline

Figure 14 displays a top down diagram of the design process and evaluation before discussing its feasibility compared to the current Ike Dike plans and reaching the final conclusions. The main items addressed along the report are presented and classified into groups corresponding to the chapter they are addressed.

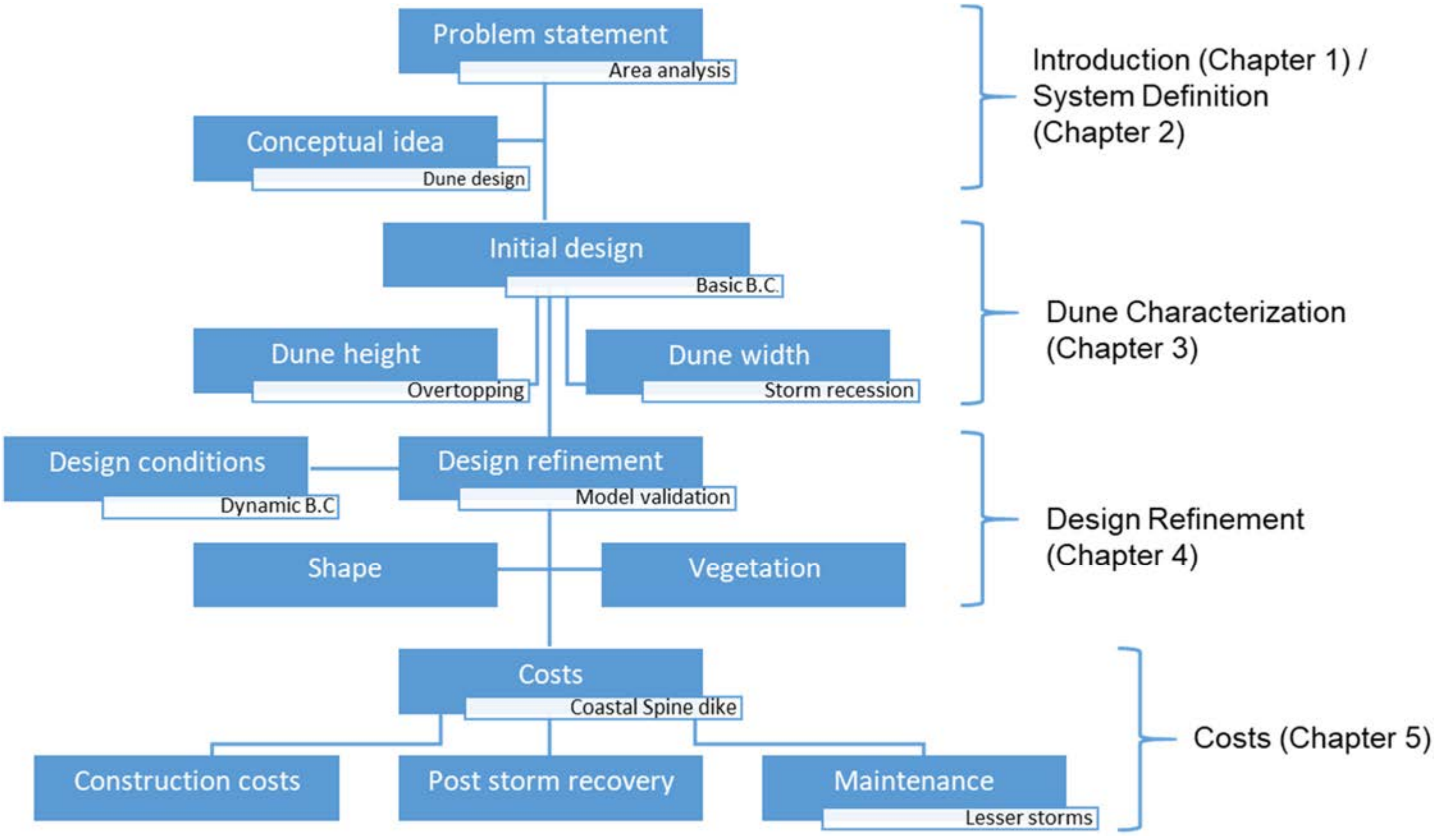

Figure 14. Thesis design process and structure diagram.

Following the workflow presented in Figure 14, the thesis structure of the thesis is as follows:

2. System Definition

2.1. Coastal Spine

The Coastal Spine or Ike Dike, as introduced in Chapter 1, is the main and preferred alternative considered for future plans regarding active flood protection in Galveston Bay and it is currently under design development (Merrell, Figlus, & van Kuijk, 2019) with different stakeholders taking part in the process due to the vast complexity and reach of the plan. Since the plan is already under development, with the first steps leading into its execution already taken (Galyean, 2018), Ike Dike will be considered from now on the default solution to reflect on in this study in order to provide meaningful results for the future plans.. It is based on the principle of the Delta Works plan established in The Netherlands of shortening the coast, thus minimizing the length to be protected and maximising the efficiency of the implemented measure.

The idea behind the Ike Dike concept is to place a dike system interconnected along the coast of Galveston Bay, running along Galveston Island, Bolivar Peninsula and connected to higher lands in their west and east ends. To achieve full closure of the bay from Gulf of Mexico during a hurricane, both tidal inlets San Luis Pass and Bolivar Roads are planned to be closed with storm surge barriers, while during normal conditions Bolivar

Roads needs to be open because it is the access point to the Port of Houston via Houston Shipping Channel. This complex and particular scenario is intended to be achieved through a combination of two Dutch Delta Works interventions: Eastern Scheldt storm surge barrier and Maeslantkering. A conceptual design of the storm surge barrier at Bolivar Roads inlet is depicted in Figure 15 Since achieving bay isolation during storm condition without the presence of the surge barriers is impossible due to the high amount of water inflow into the bay through the inlets, this thesis alternative takes the future scenario where they are constructed as a reference.

Figure 15. Bolivar Roads closure works conceptual design. Source: U.S. Army Corps of Engineers.

Since this thesis focus is on developing an alternative to the land barrier aspect of the plan, a comparison between the designed dune system and the future scenario instead of to the present situation is made along the thesis report. The most recent, developed and advanced report in the topic is *E.C. van Berchum et al., 2016* (Figure 16), thus it is the reference used to determine the feasibility of the proposed dune based alternative. Even though Ike Dike is still under discussion after 12 years, through this time progress has been made and its implementation is likely to happen in the near future (Houston Chronicle, 2018).

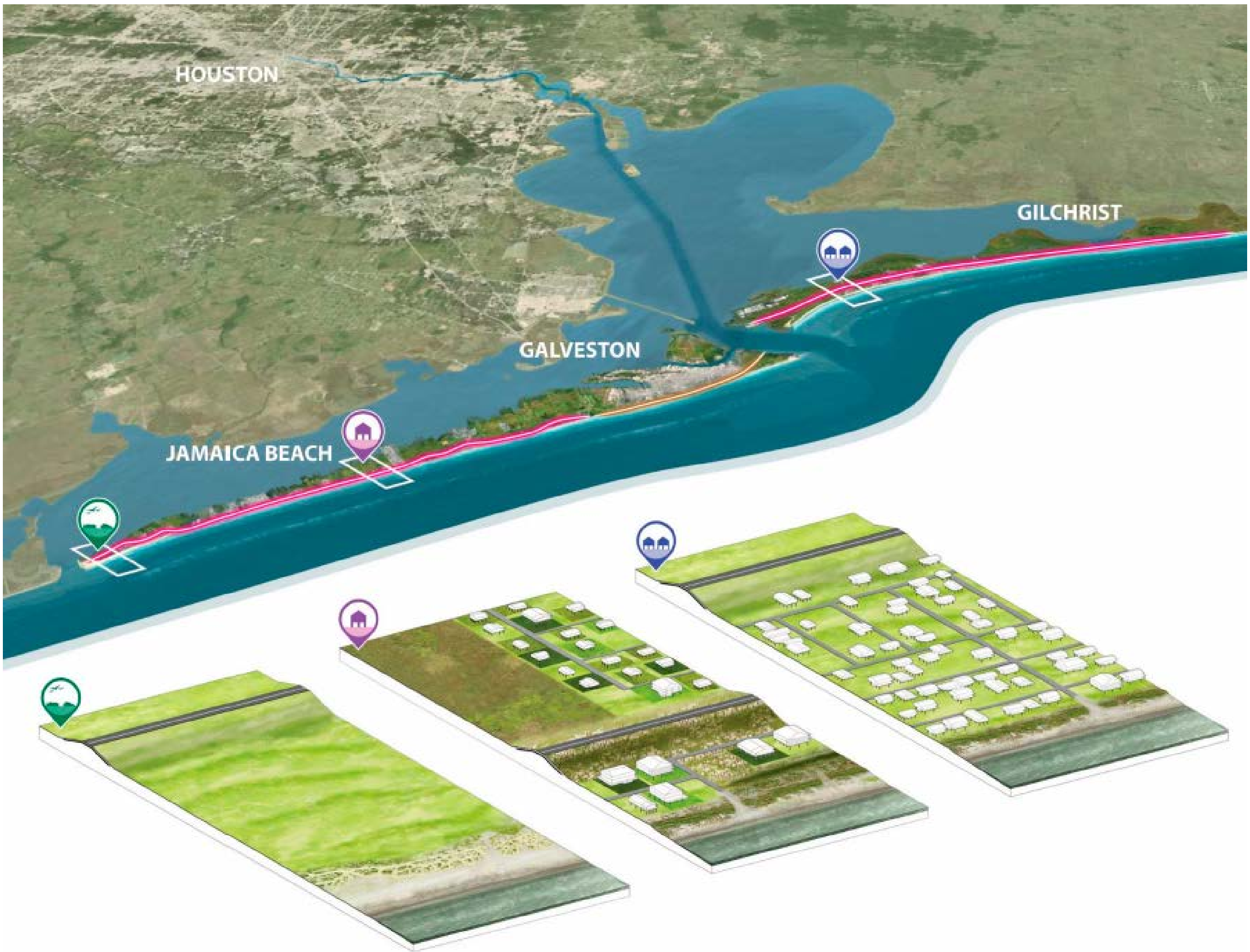

Figure 16. Land barrier conceptual design presented in E.C. van Berchum et al., 2016.

As it can be seen from the illustrations of Figure 16 and in the cross section shown in Figure 17, the planned dike is located where the main access coastal roads of Galveston Island and Bolivar Peninsula (Highway FM3005 and SH87 respectively) are running, although a dike on the beach was also considered (Figure 17) but not chosen as preferred. On the contrary, in this thesis design is proposed a placement of a soft dune at the back of the beach, where natural dunes of maximum 1.5 metres can be found at some locations along the coast (Howard, Laverty, & Ekstrom, 2013).

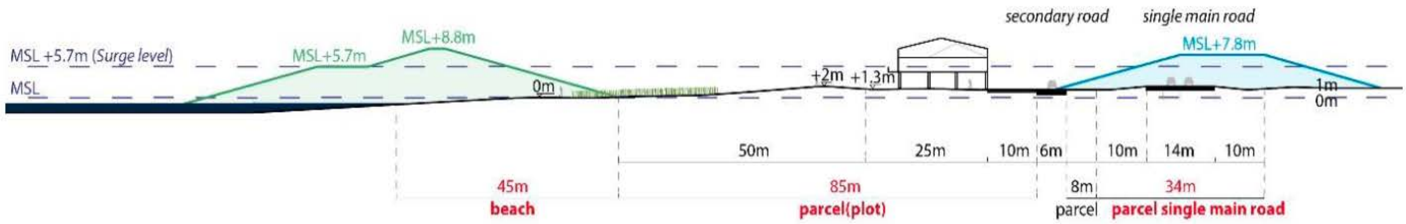

Figure 17. Two dike alternative locations presented in E.C. van Berchum et al., 2016.

Another main difference between the dune design and the dike one presented in the mentioned report is the purely sandy composition of the former one, promoting its integration within the coastal system. However, the proposed natural alterative carries consequences due to different factors such as its erodible nature that will be addressed and quantified in later chapters.

2.2. Boundary Conditions

In the U.S., the strategy to face flooding is in the mitigation and evacuation side (active) as opposed to the prevention risk based one (passive). The former approach allows for a flood protection structure design with less return period requirement, but compensating it by taking advantage of other tools like early warning systems and insurance policies. It does not mean that is less safe, but that the system is designed to give relatively more relevance to active protection mechanisms instead of passive ones.

Figure 18. Flood safety standard in The Netherlands, based on failure probabilities per dike sections. Source: (Ministerie van Infrastructuur en Milieu., 2016)

An example of a passive flood prevention system is found in The Netherlands (Warner, Weijs, & Wojiciechowska, 2012), which historically has been following a flood defence plan consisting on the division of the national territory into dike rings with different flood protection standards, some of them reaching a safety protection against a 1/10.000 year event. However, since 2017 the flood design standard is the failure probability of different dike trajectories (Ministerie van Infrastructuur en Milieu., 2016), minimizing the individual risk of the different areas.

On the other hand Galveston Seawall follows a state normative safety level against flooding of 1/100 $year^{-1}$, a value which would be insufficient in a passive flood protection scheme such as the one planned to achieve with the bay closure implementation with the Ike Dike. Furthermore, in the particular case of Galveston, the current seawall protects only the city area from hurricane action in the ocean side, leaving the possibility of back side flooding from the bay due to surge development inside the bay as the hurricane follows its spatial track. Strengthening of the land barrier is thus a requirement for future plans, as well as the extension of the barrier along the whole Galveston Bay coast. Galveston Seawall has been the main line of defence of Galveston since early XX century, when after the devastating event of the Great Storm of 1900 it was decided that Galveston City would have its elevation raised and the Galveston Seawall would be built between the city and the shoreline (Figure 19) in order to prevent the dramatic situation to be repeated again (Galveston and Texas History Center, 2004).

Figure 19. Galveston Seawall in 1904 after its construction. Source: Galveston and Texas History Center

The Galveston Sewall crest is at an elevation of 5.2 m, which is the same as the 100 year return period water level (Rippi, 2014). The presence of the seawall makes the design of the dune along the coast extent in which it is present to deviate from the two design solution, however, it is still positive since a dike-in-dune concept can be applied in order to implement a homogeneous solution along the coast.

The storm conditions for different hurricane return periods can be seen in Table 2 (Jin et al., 2010; van Berchum et al., 2015). The daily basis conditions are shown in Table 3 (de Vries, 2014).

	$1/100\ yr^{-1}$	$1/1{,}000\ yr^{-1}$	$1/10{,}000\ yr^{-1}$
Maximum Surge [m]	**5.2**	6.1	7.0
Offshore Significant Wave Height H_s [m]	**5.0**	5.7	6.3

Table 2. Hurricane hydraulic boundary conditions for different return periods.

Low Tide [MSL+ m]	-0.25
High Tide [MSL+ m]	0.39
Significant Wave Height H_s [m]	0.5
Peak Wave Period T_p [s]	4.0

Table 3. Regular hydraulic boundary conditions from the Gulf of Mexico.

The bathymetry and topography digital elevation map (DEM) is obtained from the National Centers for Environmental Information (NOAA, 2006) (Figure 20). With it, the bed level and topography of different cross sections can be obtained via GIS tools.

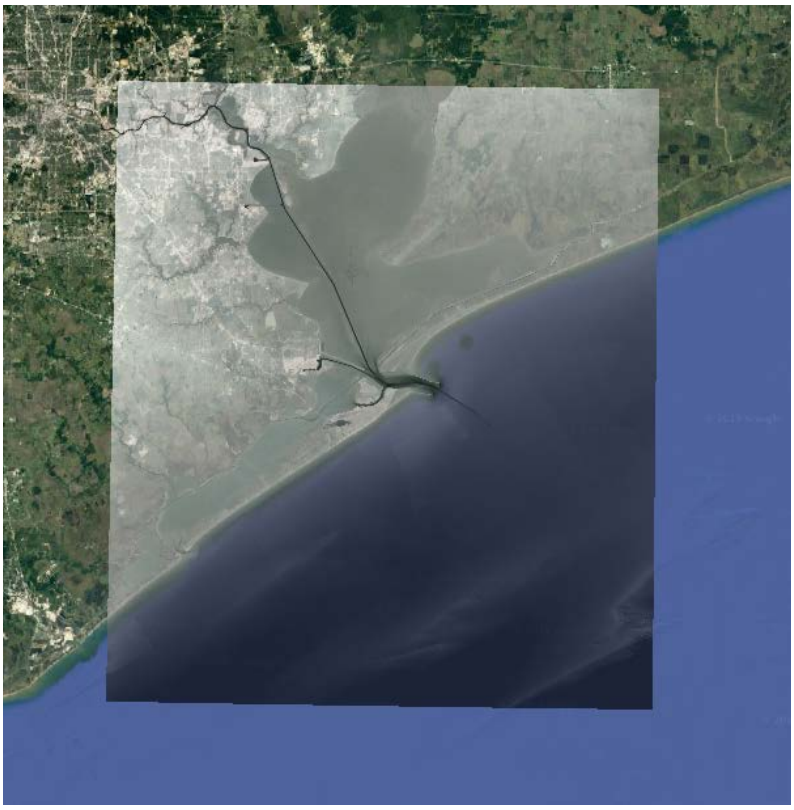

Figure 20. Overlaid DEM (grey) of Galveston Bay and site map. Source: (NOAA, 2006)

2.3. Area Spatial Characteristics

The Galveston Island and Bolivar Peninsula area features differ significantly from part to part. Galveston Island is the most populated of the two with around 50,000 inhabitant only in Galveston City versus 2,700 inhabitants in Bolivar Peninsula in 2018 (United States Census Bureau). However, the western end of Galveston Island is the protected natural area of Brazoria Natural Wildlife Refuge. As stated in Chapter 1, the Port of Houston account for a vast vessel activity with a wide variety of cargo, all of them crossing the Bolivar Road inlet via Houston Shipping Channel, with its deepened bottom clearly visible at Figure 20. This makes its closest areas to experience more nuisance from this activity from the generation of extra waves or an increase of pollution.

On the other hand, Bolivar Peninsula does present heavily urbanized areas such as Galveston City. However, along the peninsula there are present sparse residential communities such as Port Bolivar, Crystal Beach, Caplen, Gilchrist or High Island (Figure 21). The east of Rollover Pass is bordering the Anahuac National Wildlife Refuge conservation area.

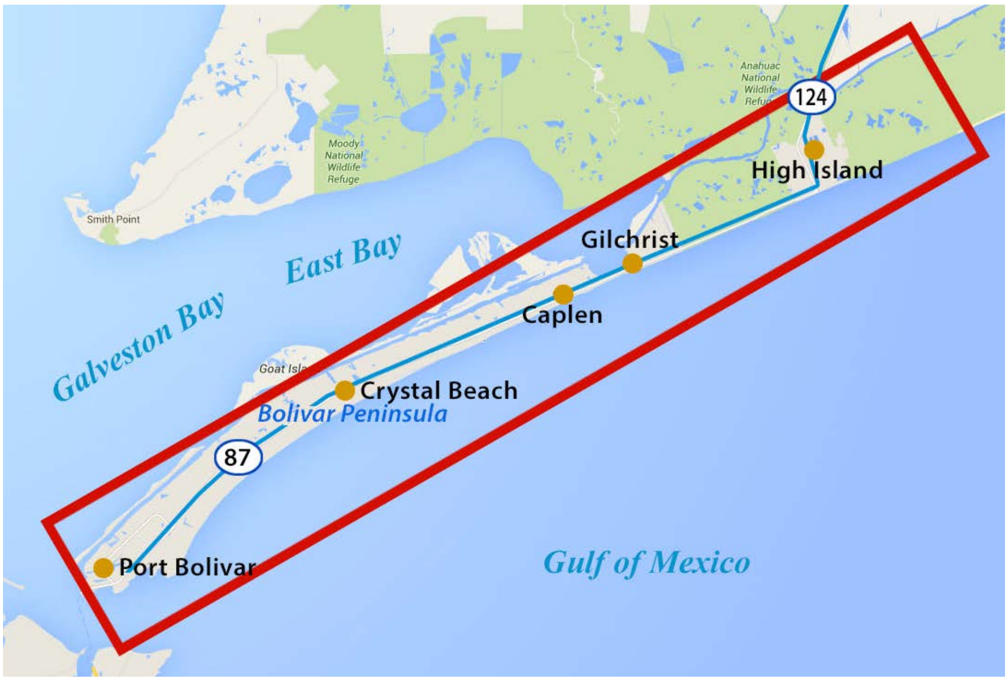

Figure 21. Bolivar Peninsula residential communities

The presence of the explained heterogeneities within the case study area makes tough the design process of a general solution since it is unlikely to find a solution that fits all. The initial intention for the future flood defence system regarding the land barrier was first to take the concept of the current Galveston Seawall and expand it across the whole coastline (van Berchum et al., 2015), and later modified to a dike solution placed following the coastal highway paths (van Berchum et al., 2016) with the purpose of reaching the bay closure in an event scenario in this way. However, not every coast reach is suitable in the same degree to this intervention, especially those with a more heavily natural land use focus.

The soft solution proposed in this thesis has a certain degree of flexibility due to its sandy nature that makes it suitable to any coastal environment along Galveston Bay coast.

2.4. Dune Concept

The proposal developed in this thesis consists on a continuous line of defence placed at the back of the beach along the entirety of the Galveston Island – Bolivar Peninsula coast. The dune design for is inspired by a land barrier concept alternatives in early Ike Dike reports (van Berchum et al., 2015). Specifically, the one depicted in Figure 22. However, some differences apply from this concept to the one proposed in this report.

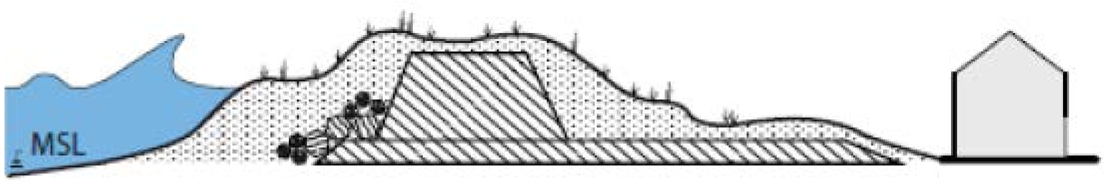

Figure 22. Concept sketch of dike-in-dune. Source: (van Berchum et al., 2015)

One of the main differences with the idea presented here is that there is no hard structure involved. The buried dike showed in Figure 22**¡Error! No se encuentra el origen de la referencia.** would not be present in this thesis dune design, it is substituted with a sandy core. Nonetheless, Figure 22 is a reliable visualization of how the dune system would be implemented where the Galveston Seawall stands, since its presence makes it more convenient to follow the dike-in-dune approach.

A visual representation of this conceptual design, comprising a representative section where the standard dune solution would be implemented is depicted in Figure 23.

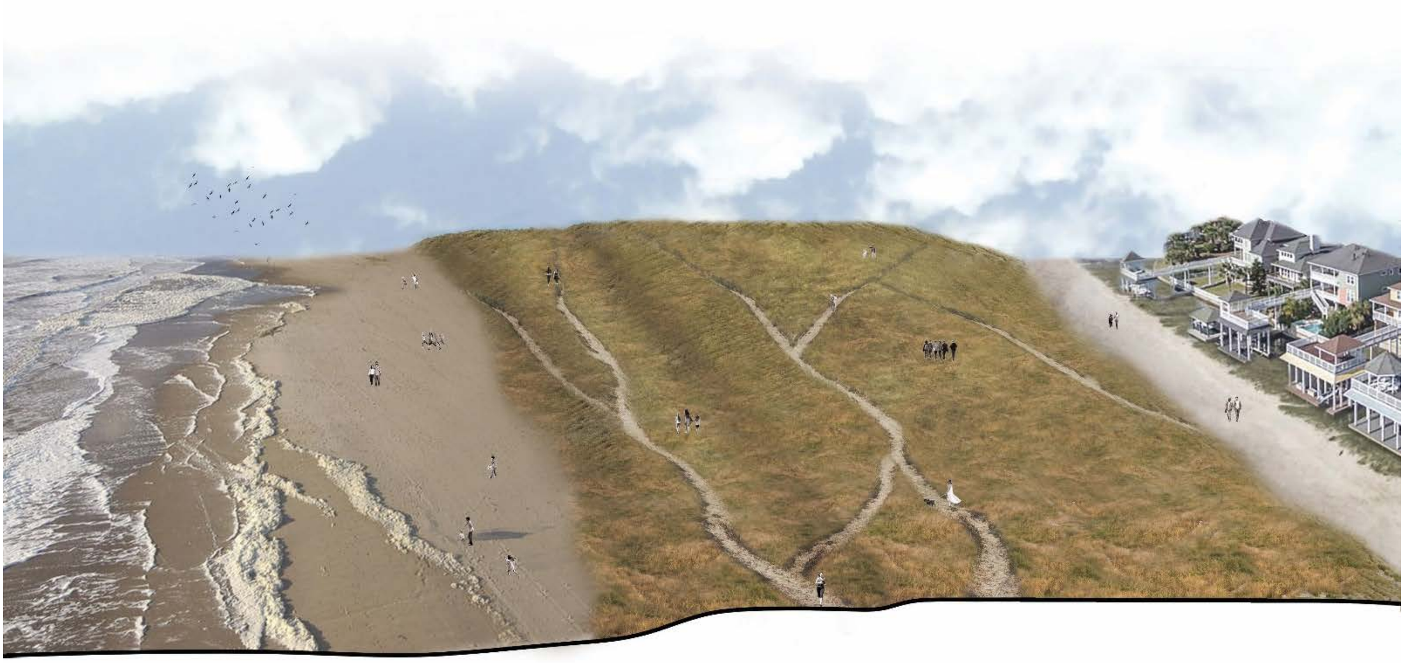

Figure 23. Illustration of the traditional flood protection dune concept design.

The design must be dimensioned correctly to ensure its capability of withstanding the hydraulic forcing caused by the design 1/100 yr^{-1} hurricane since that is the previously established design boundary condition. The situation and profile development expected and aimed for is visually represented in Figure 24.

Figure 24. Illustration of the traditional flood protection dune at the beginning (left) and during the storm (right).

The concept behind this thesis alternative design is to protect against a coastal flooding situation like a dike would do, but in a daily condition provide quality of life improvements. For example, it provides to the coast the ability of retreat along with the shoreline in a natural dynamic process if needed. With a hard structure preventing beach retreat, the coast disappears with time, as explained in Chapter 1. In order to prevent the beach from disappearing, nourishment maintenance is required. Conceptual art of how Galveston Bay could look like with an extended seawall along the whole coast with the mentioned ongoing beach erosion process is presented in Figure 25. The differences between Figure 23 and Figure 25 in terms of landscape integration, accessibility and ecology features are clearly noticeable, removing the sudden transition from coastal to inland areas caused by the hard structure placement.

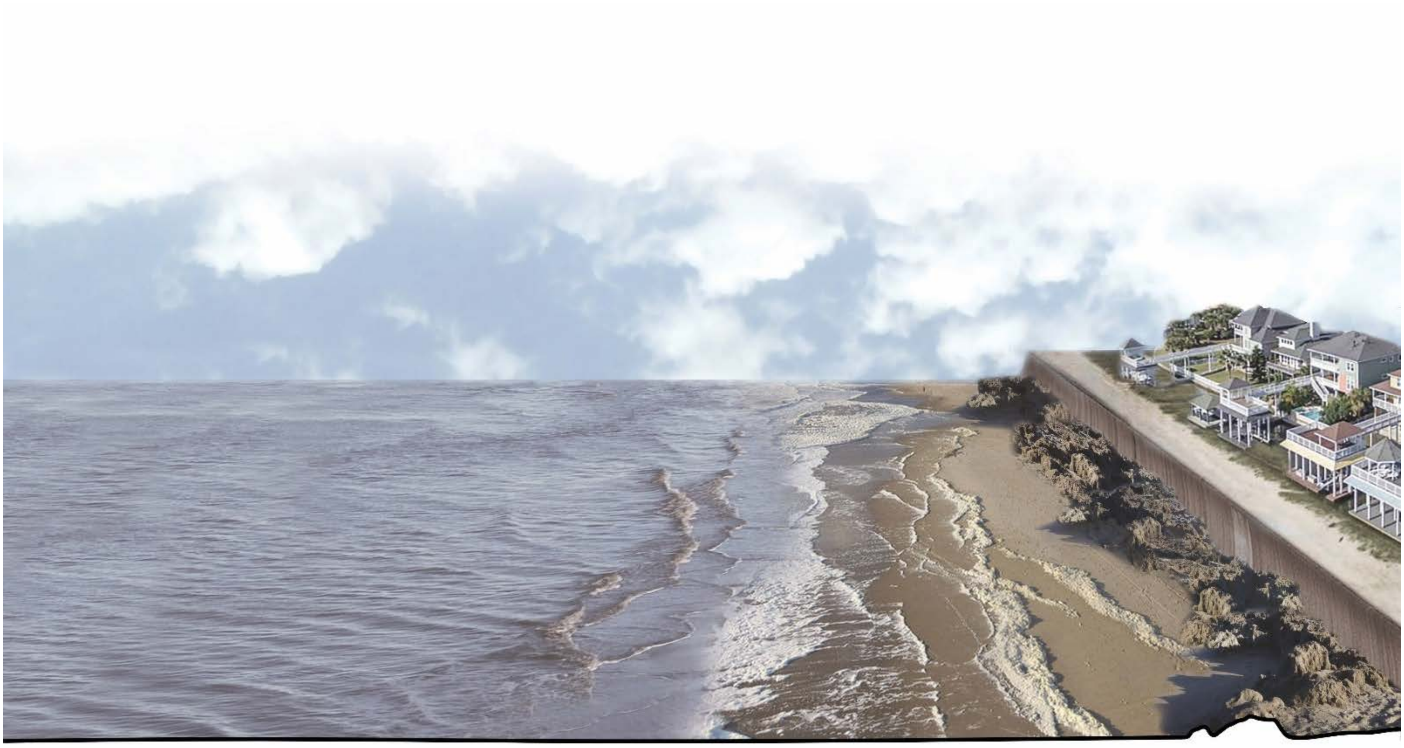

Figure 25. Illustration of the possible extended seawall intervention.

Another factor in favour of the dune design to take into account is the flexibility of the intervention proposed in this thesis. If a hard structure like a seawall or dike is to be constructed, it will remain in the system indefinitely, requiring heavy work to be removed. A sandy solution can better adapt into the system if in the future any change is wants to be implemented or any complementary action taken since quality dune sand is a valuable coastal resource that can be reused in later interventions such as nourishments.

3. Dune Characterization

In this chapter the defining design parameters dimensioning the dune will be obtained through an initial quantification of the physical processes that govern each of them. Dune shape is determined by its slopes, which need to have an angle that ensures its stability. Crest height is obtained through an overtopping analysis in a way that limits the amount of discharge over the dune during design storm conditions. Finally, dune width is estimated by predicting the recession the dune experiences over the hurricane scenario and accounting for a safety margin that provides sufficient reliability to the system.

3.1. Dune Shape

The shape of the dune will be determined by the slopes of its inner and outer side. The gentler, the more stable it is and the less risk of collapsing. However, the volume of the dune greatly increases as well as the space needed with a milder slope. This may pose a problem due to the scarce quality sand availability in the zone (Anderson, 2007).

The slopes will be covered with a vegetation layer, which provides two main engineering benefits: it helps improving the stability of the dune and it is an overtopping reduction factor, which will be discussed in the next section. Furthermore, the use of vegetation from the area such as panicum, morning glory or sea purslane provides a natural look and integrates to a high degree with the ecosystem (Figure 26).

Figure 26. Vegetated dunes at Galveston Island State Park. Source: (http://www.galvestonislandstatepark.org)

The overall stability is determined by the material, in this case sand. Based on its angle of internal friction, 30º in this case (Eurocode 7 NEN-EN9997), the same slope would suffice to provide an stable situation (Tiwari, Sharma, & Yadav, 2016). However, there are some points leading to a safer slope.

The main argument is the variability within the sand characterization. The median diameter D_{50} of Galveston sand is 0.132 mm, and together with its grain size distribution (D_{90} is 0.1869 mm) makes it a relatively fine sand (Lisle & Comer, 2011). This calls for a milder slope use, with a starting value of a **1:3 slope** (18.43º above horizontal). In later stages of the design, this value may change if the conditions allow for it.

3.2. Crest Height

Overtopping is the main parameter governing and determining the height of the dune. This overtopping can cause a rapid and critical erosion of the dune at the front, crest and back if it is prolonged over a relatively long period of time; potentially leading to stability loss or the complete dune wash away if the intensity is high enough.

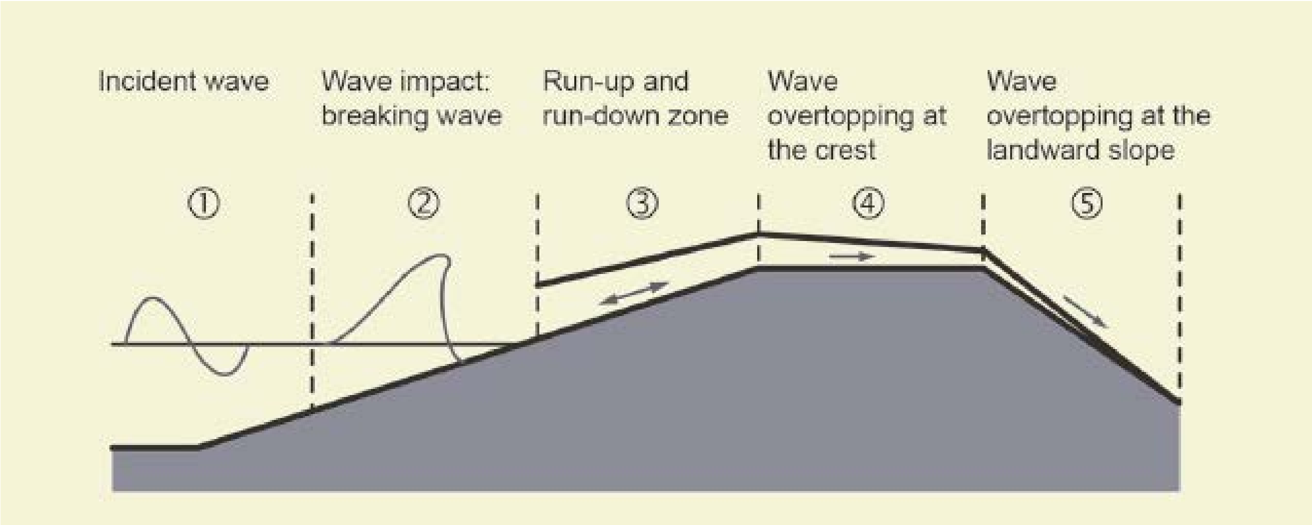

Figure 27. Coastal process leading to wave overtopping. Source: (Van der Meer et al., 2016)

The overtopping estimation will be performed following the guideline provided by the Overtopping Manual (Van der Meer et al., 2016). Although the first design offshore significant wave height has been initially established as 5.0 m, due to depth induced breaking, the design wave height is lowered to 3.2 m after the results yield by SWAN modelling (van Berchum et al., 2016). The overtopping rate is estimated for three different dune crest levels: 5.2 m (matching the 1/100 water level), 7.5 m and 10 m. The process followed to obtain the discussed results is found in Appendix A.

The most relevant results are presented in Table 4. One relevant parameter influencing overtopping discharge is wave period. As an initial estimate (in Chapter 4 it will be developed in the model), hurricane Ike will serve as a reference to choose an appropriate wave period due to the relative proximity of its boundary conditions to the design ones. Looking at past hurricane data values (Wu, Taylor, Chen, & Shaffer, 2003) is a good way to obtain a first design hurricane wave period due to its independency to the rest of the parameters, especially to wave height. It is seen how category 4 hurricanes like Ike, generate waves with peaks around 14 s, thus this value will be taken as the initial one. Later on it can be verified if this initial estimate is in agreement with the recorded data.

Crest level [+MSL]	5.2 m	7.5 m	10 m
Overtopping discharge [$m^3/m/s$]	0.649	0.0443	0.0006

Table 4. Overtopping results for different dune crest heights.

As it can be seen at first glance, the discharges and flow velocities are drastically reduced from one case to another. An early conclusion is that a raise of the dune of around 2.5 m leads to a reduction of overtopping discharge of an order of magnitude of 10 the first time and 100 the second one. With the objective of selecting an adequate dune height, a table from the overtopping design guidelines (Van der Meer et al., 2016) indicating a range of design overtopping rates and their expected respective consequences in the structure is presented (Table 5).

Mean overtopping discharge [l/m/s]	Qualitative damages
1	Limited damage to inner slope
10	No damage to crest and rear face of grass covered embankment of clay
50	Limited damage to crest and rear face of grass covered clay embankment
200	No damage if crest and rear slope are well protected
600	Extreme overtopping, major damage is accepted after a 1/100 yr^{-1} storm. Additional measures required for drainage on rear side.

Table 5. Expected qualitative damages for different overtopping discharges. Source: (van Berchum et al., 2016)

Although, as it can be seen from Table 5, limiting as much as possible the overtopping associated damages is crucial for the design safety, the plan needs to remain feasible and be coherent with the case limitations. Raising a 10 m high barrier at the coast, even if it is a dune based one and would be extremely safe regarding overtopping damages, would cause a shock in the landscape and society while also elevating the cost of the project significantly. On the other hand, planning a dune at the same design water surge level would not be the most efficient idea since overwash during storm conditions would compromise severely its flood defence functionality.

The MSL +7.5 m crest option yields a result of 44.3 l/m/s, a value that lies in the category where limited damage is expected as long as the structure is covered in grass, aspect that will be implemented and discussed in Chapter 4. Although damage is still expected, the overtopping estimation was performed using the design boundary conditions, which during hurricane scenario are expected to happen only during its peak and not be prolonged over time for long.

Due to the calculations performed, analysis of the results and discussion of the consequences, it is established that the initial design crest height is **MSL +7.5 m** and that, if necessary, in the next chapter this value can be adjusted attending to future challenges to further optimise the design.

3.3. Dune Width

Dune recession associated with dune storm erosion (Figure 28) taking place during a hurricane event is the parameter that determines the width of the dune. Along the development of a storm the design dune is constantly being eroded, with the most critical spot around water level height, where incoming waves are making impact with the dune front. Erosion values escalate with the storm intensity (storm surge and wave characteristics) and is also affected by bed properties (slope and grain size) (van Rijn, 2013).

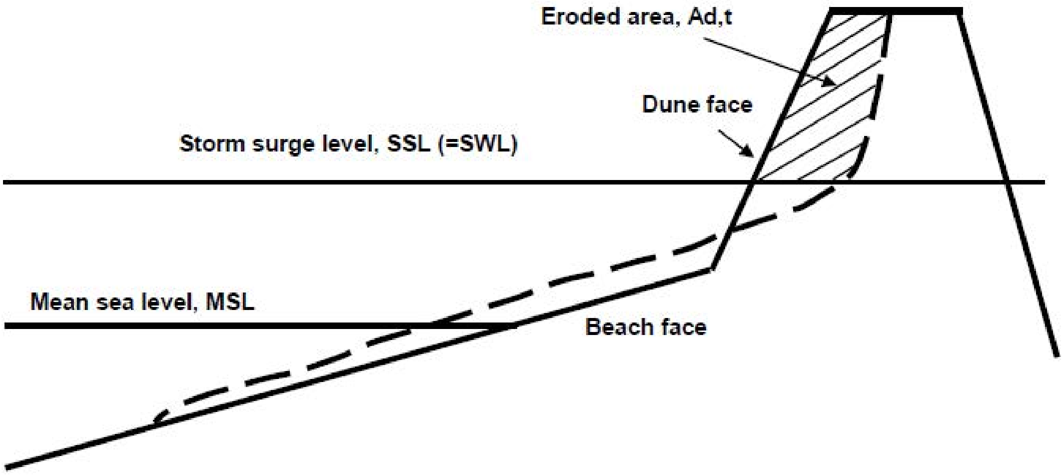

Figure 28. Sketch of dune erosion due to a storm. Source: (van Rijn, 2013)

Dune recession as a parameters determining dune width has to be taken as an initial reference since the failure of the dune tends to happen before due to either breaching or crest/back erosion. For example, before the theoretical erosion value takes place the dune has already collapsed due to instability caused by erosion.

The 'Simplified dune erosion rule' (van Rijn, 2013) is the chosen method to obtain a first dune recession value using the parameters that influence the process, as it has been proven able to provide with a relatively accurate dune recession value in agreement with field cases as well as monitored laboratory tests. The method has been developed based on CROSMOR model runs, obtaining a simplified equation. This way, with some site parameters it is possible to estimate an initial value to keep on working with. The procedure of obtaining the results can be followed in Appendix B.

The average horizontal dune recession at storm surge level from the design 1/100 storm with incident waves attacking the front for 2 hours is 54.65 m. This means that extrapolating the results of the testing models from which the erosion rule is derived, at the 5.2 m water level mark the dune would be eroded for 54.65 m. Thus an initial width from inner to outer dune foot or base level of **100 m** (with 1:3 slopes) that accounts for safety margins is a possible starting point that can be refined later on in subsequent chapters.

The dune foot is located at MSL level, which makes it possible to use the beach profile to be part of the base of the dune. This means that the actual footprint of the dune in the beach is variably reduced, depending on the specific location making use of the local topography of the terrain.

To summarize the initial dimensions, the dune has a **base width of 100 m at MSL**, its crest is at a **height of MSL +7.5 m** and has a **1:3 slope** ration, which accounts for a **crest width of 55m**.

4. Design Refinement

4.1. Design Summary

In order to have all the information about the dune dimensions, characteristics and hurricane boundary conditions condensed, Table 6 is presented. References are made to the sections where the specific parameter is develop and its derivation procedure so it can be easily found, the initial design parameters have been estimated in Chapter 3 while the ones related to modeling are obtained through this chapter.

Dune crest (Section 3.2)	Level	MSL +7.5 m
	Width	55 m
Dune base (Section 3.3)	Level	MSL
	Width	100 m
Dune slope (Section 3.1)	1:3	
Boundary condition return period	100 years (1/100 yr^{-1} protection level)	
Storm surge	Preliminary (Section 2.2)	5.2 m
	XBeach model (Section 4.3.1)	Figure 32
Wave height	Preliminary (Section 2.2)	5 m offshore, 3.2 m at dune foot
	XBeach model (Section 4.3.2)	Figure 35
Wave period	Preliminary (Section 3.2)	14 s
	XBeach model (Section 4.3.2)	Figure 37
Dune sediment (Section 3.1)	D_{50}	0.1323 mm
	D_{90}	0.1869 mm
Vegetation (Section 4.5.2)	Location	From MSL +1 m (front slope) To MSL +4.5 m (inner slope)
	Height	0.25 m
	Stem diameter / Blade width	0.01 m
	Density	500 units/m^2
	Drag coefficient	0.4

Table 6. Dune characteristics summary.

4.2. Design Location

The entirety of the coast length needed to be protected in order to provide Galveston Bay area flooding security is in the order of 80 km, which gives an insight of the magnitude of the stretch to be protected. Designing for every section stretch accounting for each distinctive feature (beach width, residential areas, road location, seawall…) is out of the scope of the thesis. Thus, a design location is chosen attending to features that may oppose a problem in terms of the dune system execution.

Van Berchum et al. (2016) in its most recent land barrier design, categorized different Galveston Bay coastal sections into different categories (Figure 29). However, this categories have a focus on their cross section and their available space present to allocate a land barrier structure.

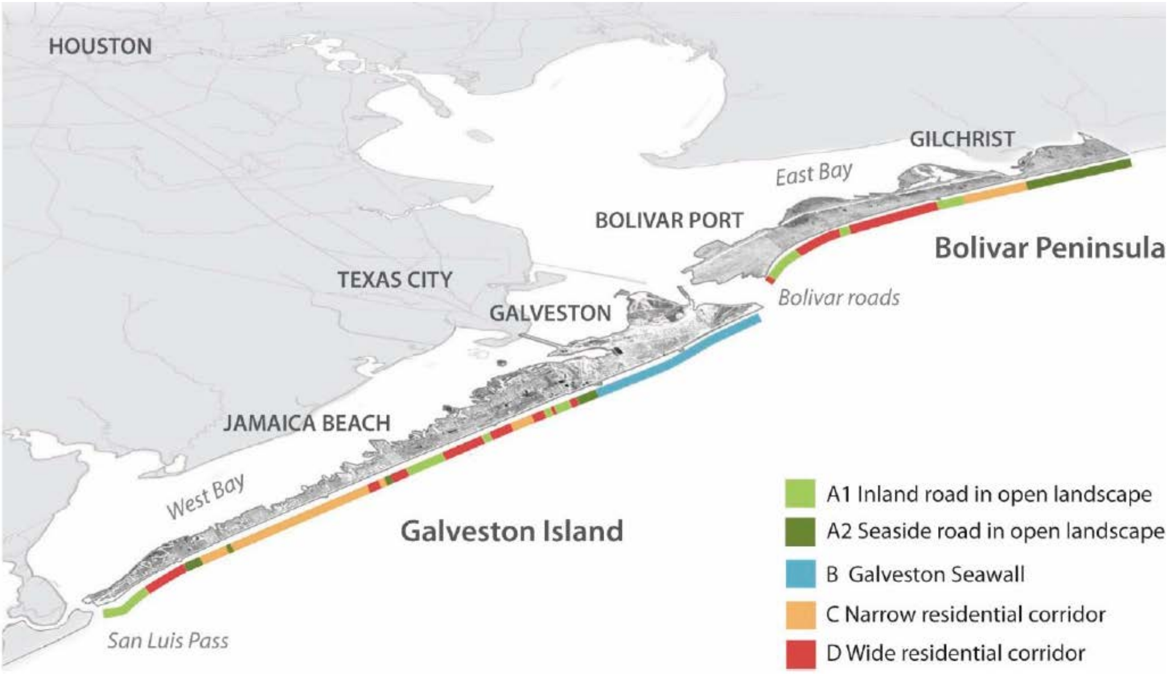

Figure 29. Characteristic Galveston Bay coastal categories. Source: (van Berchum et al., 2016)

The dune most restrictive feature, in terms of impact and feasibility, is its dimensions. It will need to reach a height of MSL +7.5m and the crest should extend to 55 m. This means that there must be sufficient space to allocate the dune. In Figure 31, the sections categorized as C (narrow residential corridor) are the ones where generally the available beach width can be a limiting factor for this design. One of the main purposes of this design is, apart from preventing storm surge to enter the bay contributing to overall bay flooding reduction, to protect in a local scale the properties directly behind the defence system without leaving any residential area at its own risk. Thus, one of the areas meeting these two criteria is selected. Jamaica Beach (Figure 30), in Galveston Island, fits the given description of a suitable design location.

Figure 30. Jamaica Beach location in Galveston Island.

Jamaica Beach is one of the communities in Galveston Island that, while not being as densely populated as Galveston City, it does not fit into rural territory. Historically, it has been one of the communities actively involved in dune protection and enhancement (Dennis & White, 1993). Its beach width ranges between 40 and 60 m, being a relatively narrow space to work on. As it can be seen in Figure 30, the residential properties extend across the whole section all the way up to the back side of the island, where the houses at the back of the island reach the bay side and are arranged along a canal system.

The area characteristics of the design location, the specific design cross section as well as its topography (NOAA, 2006) is presented in Figure 31. In it, the different characteristic land types along the cross section such as the beach, coastal road and residential areas including the canal system have been specified.

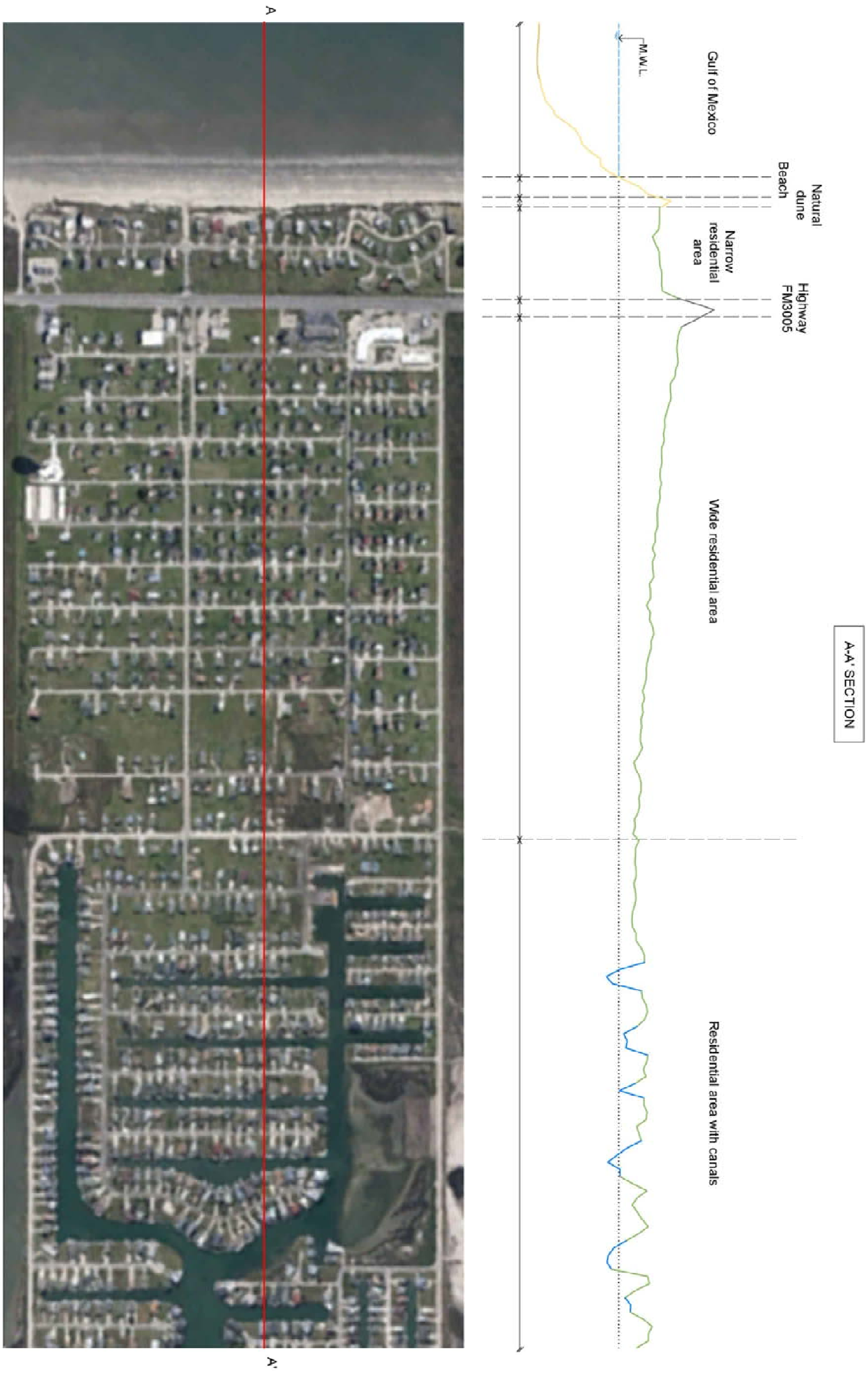

Figure 31. Design location cross section in Jamaica Beach, Galveston Island. General scale 1:7500. Cross section vertical scale 1:150.

4.3. Model Set-up

In the previous chapter the basic dimensions of the dune design have been established. In order to assess the performance of the design in a design storm scenario, numerical modelling will be used. As the situation to be evaluated is a coastal short term event, an XBeach model will be used. As explained in Section 1.4, it is the most logical tool to use since it strong point is the evaluation of coast erosion and cross shore profile change during an event due to surge and wave action processes.

To ensure the results are sufficiently reliable, the model has to be validated. Water levels and wave climate data collected by NOAA and USGS during hurricane Ike in 2008 are of major convenience since the 1/100 design storm boundary conditions don't deviate excessively from them. This fact gives the confidence that once the validation yields acceptable results, the ones from the design model can be trusted. The performed XBeach validation procedure is explained in Appendix C.

An explanation for the most important boundary condition and settings choices is given below.

4.3.1. Storm Surge

When estimating the first design dune height in the previous chapter, the 1/100 water level was assumed to be at the same level as the Galveston Seawall, 5.2 m (Rippi, 2014). This is due to the fact that similar preliminary design reports (van Berchum et al., 2015) worked with this initial highest surge value. However, later works suggest that 5.2 m might be overestimating the severity of the storm and a new value of 4.7 m was obtained via extreme value analysis (Almarshed, 2015).

Even though the boundary condition from which the first dune height has been derived is 0.5 m higher than the latest studies suggest (4.7 m), this provides the design an initial robustness and safety margin which could be changed at a later stage depending on the modelling results if necessary.

The goal of this part is to model the whole storm duration to obtain post-hurricane results after a realistic scenario, variation in the surge level over time has to be taken into account and implemented. This way not only the storm during its peak will be the only forcing affecting the erosion, but also the rising, forerunner and decrease of the water level. The whole duration of the storm and its development is the cause for the dune profile change.

Replicating the 1/100 offshore storm water level time series can be done through the one measured offshore during hurricane Ike at Gulf of Mexico (van Berchum & Mobley, 2017). In terms of storm surge, the later did not reach the 1/100 design condition of 4.7 m peak water level, being 4.0 m its highest level. However, in order to assume realistic values, its water level time series can be taken as reference for the design boundary condition of the design case. The ratio between the highest water level of both design and Ike scenarios (4.7/4.0) is applied to the whole Ike water level range to obtain the 1/100 design offshore storm water level profile. The result can be seen in Figure 32.

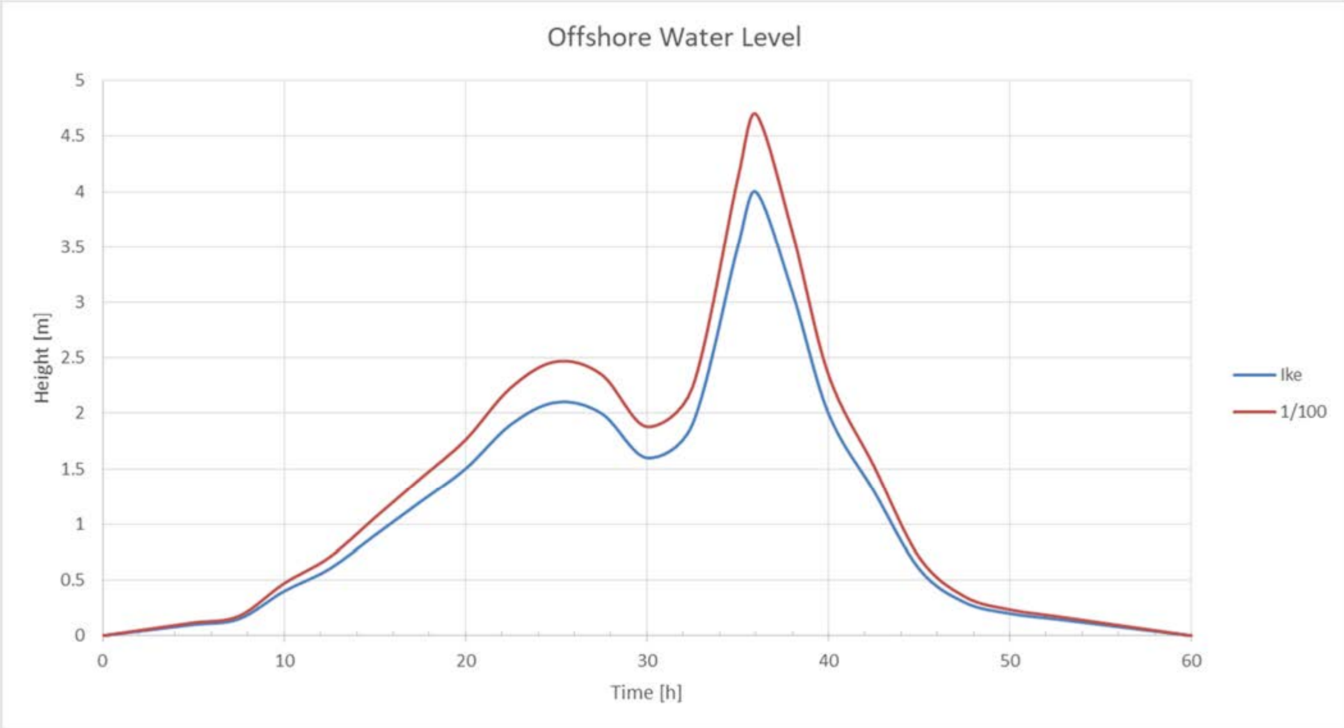

Figure 32. Measured Ike water level at Gulf of Mexico and design 1/100 offshore water level.

Although a relatively large and unpredicted forerunner is one of the Ike distinctive features (Kennedy et al., 2011), it is not wrong to assume that a 1/100 storm can have a similar water level variation profile. Past hurricanes of similar magnitude shared this same behaviour characteristic of areas with wide and shallow continental shelves such as the one seaward of Galveston Bay. Since the design location of the storm is the same, it is sensible to assume a similar water level development. Furthermore, this means providing yet another layer of security in the design by accounting for a not favourable surge development in the model.

Lastly, it has been assumed that the design duration during which the water level surge variation remains affected by the hurricane. There is no clear reason to believe that a storm with a slightly higher return period will last longer. However, the previously stated abnormally long forerunner is a factor that stretches the hurricane impact duration. This fact acts as a correction for the possible extended design storm duration.

4.3.2. Wave Characteristics

As it happens with surge level, wave conditions vary over the duration of the storm. Wave measurements are taken from the buoy Station 42035 (Figure 33), located 32 km offshore from Galveston Bay coast, consistent with the offshore water level boundary condition. Following the same procedure as for storm surge in the previous section, the measured significant wave height before, during and after hurricane Ike (NOAA, 2008) (Figure 34) will be taken as a representative sample from which the 1/100 values will be obtained.

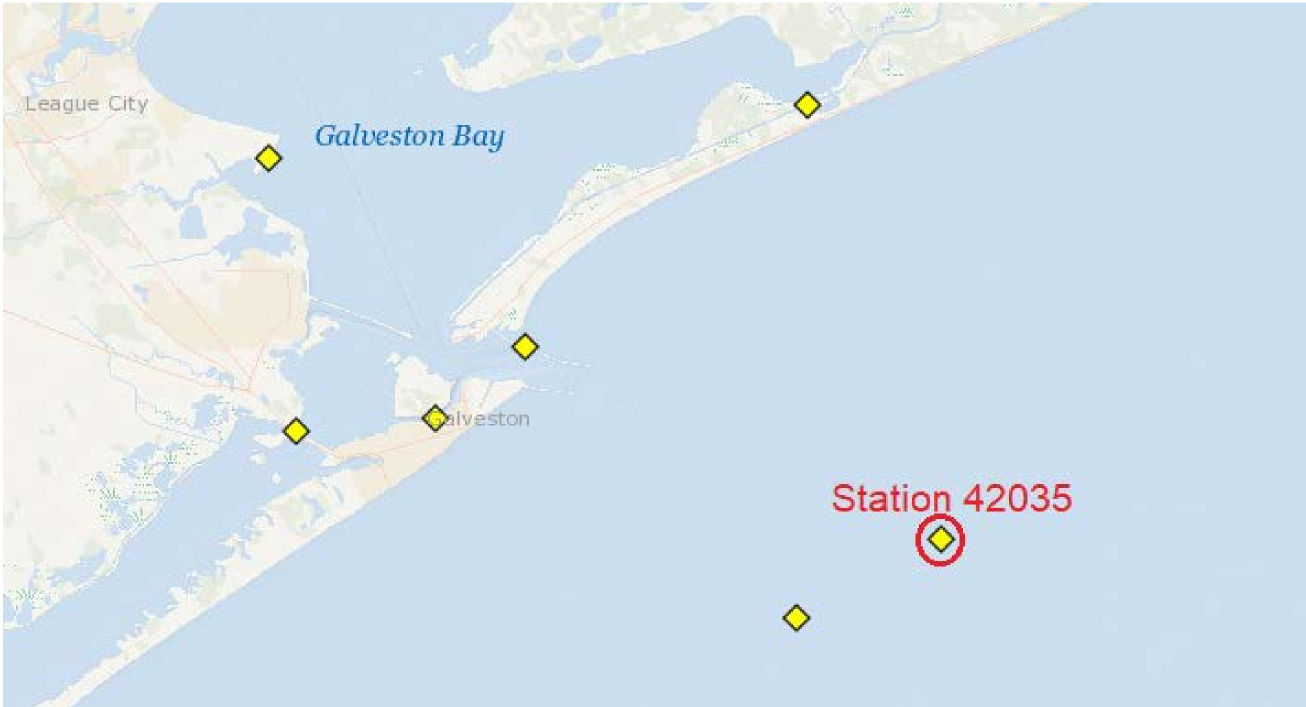

Figure 33. Station 42035 location. Source: (NOAA, 2008)

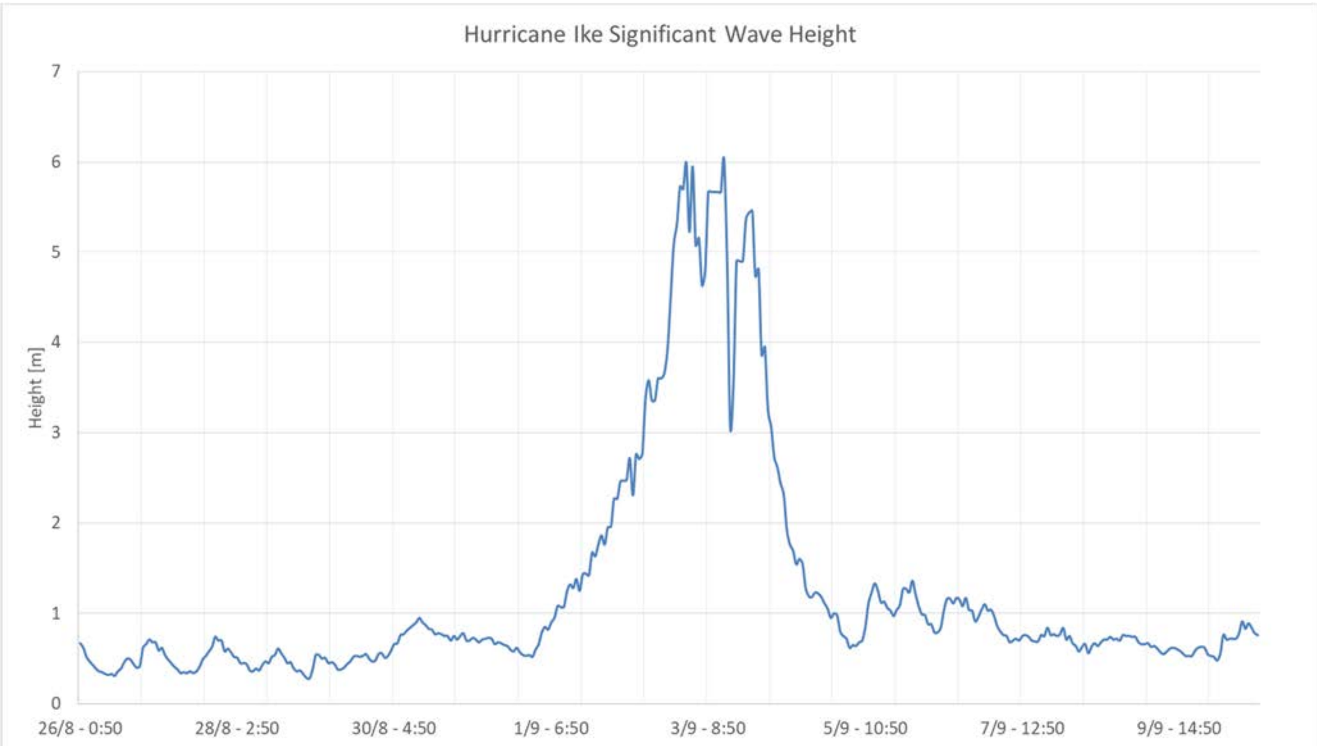

Figure 34. Ike significant wave height time series measured at Station 42035.

Following the previously applied procedure with the storm water level, the relation between the offshore design wave height (6.9 m) and the highest record during Ike (6.03 m) is applied to the whole time series (Figure 35). This method should give a reasonable and sensible estimation of the design conditions since, although being less severe, Ike is relatively close to the design scenario in terms surge and wave height. Thus, having Ike as a reference gives a starting scenario that reduces the 1/100 situation uncertainty.

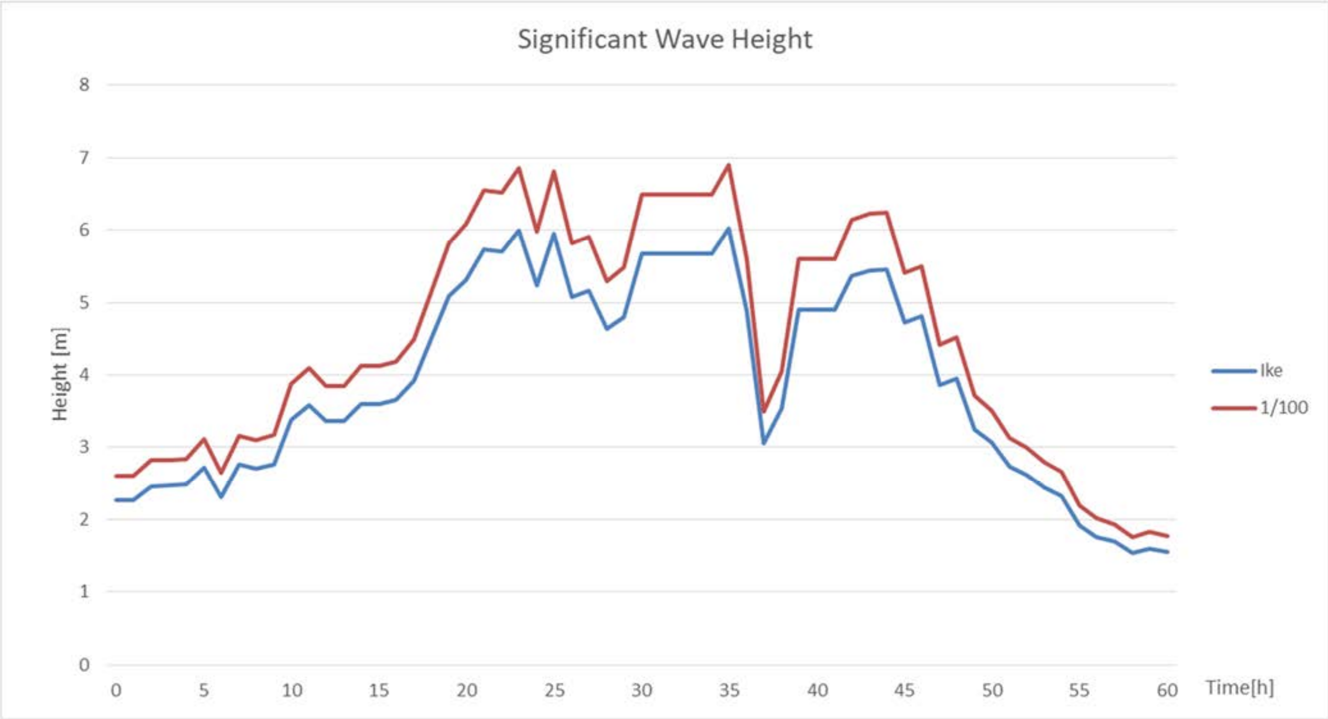

Figure 35. Measured Ike wave height at Station 42035 and design wave height time series.

The other main input wave parameter that determines the design is wave period. Unlike what has been done with wave height, there will be no transformation in this case. The same peak period measured during hurricane Ike at Station 42035 (NOAA, 2008) (Figure 36) will be used as input for the model. The lack of relationship between wave height and other parameters with its period, is the basis of using the same peak period. However, as it can be seen in Figure 36, there is a wide spectrum of wave period values taking place during Hurricane Ike life span, ranging from 4 s up to 16 s.

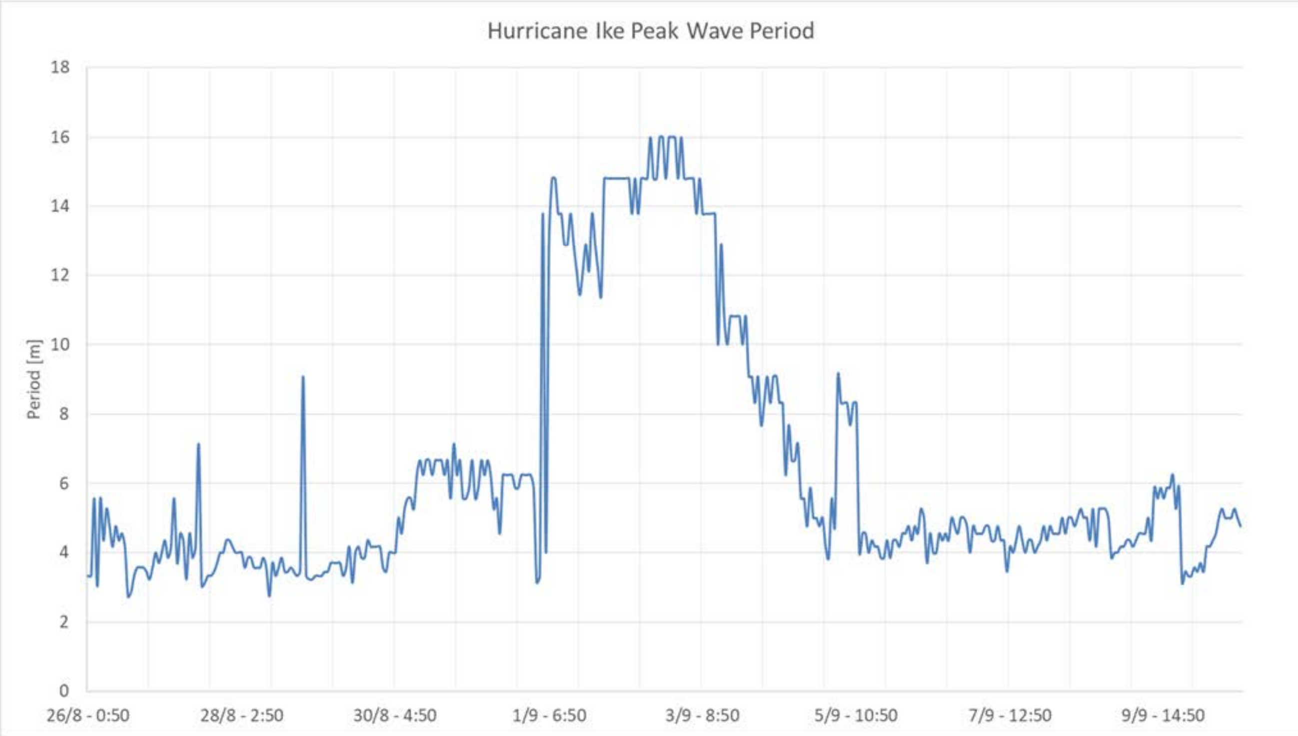

Figure 36. Wave peak period during hurricane Ike measured at Station 42035.

The peak period values observed during the time of Ike impact (Figure 37) will be the ones used as XBeach input as they can be seen as representative for a relatively similar category impact. In Chapter 3 an initial design wave period of 14 s was used, as it can be seen at this point, the initial estimate is within the 2 s range from the most critical wave period (16 s) during Ike and is indeed a relatively accurate characteristic value.

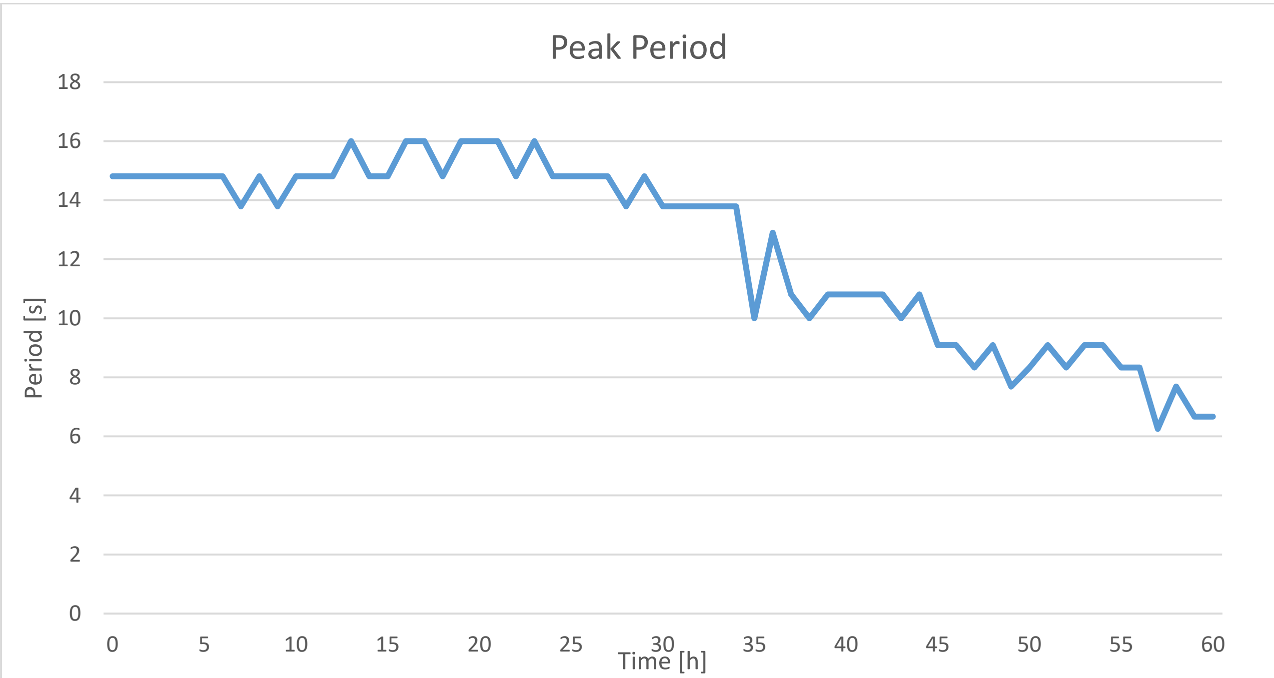

Figure 37. Design peak period, with a maximum value of 16 s.

4.4. Design Parameters Modelling

The XBeach run described and analysed in this section is the one containing the combination of dune variable parameters (dune height, width and slope angles) that resulted in the most promising results. The process of concluding which one is the combination of design values that can be taken as the most acceptable design took several iterations. The qualitative results and conclusions obtained from the most representative runs are summarized in Table 7. The run selected as the one with the most satisfactory outcome is defined by the same design values shown in Table 6 (dune crest at MSL +7.5 m, base dune width of 100 m and dune slopes with 1:3 ratio), run 3 in Table 7. The reasoning leading to the conclusion that those are the parameters comprising the final design is found in Section 4.4.5.

Run	Dune width at MSL [m]	Crest height [MSL +m]	Slope ratio [-]	Result conclusion
1	100	5.2	1:3	Dune collapses, crest eroded and dune washed away by overflow
2	100	6.5	1:3	Dune holds, excessive overtopping
3	**100**	**7.5**	**1:3**	**Dune holds, acceptable safety margin (post storm profile)**
4	100	9	1:3	Dune holds, negligible overtopping
5	85	7.5	1:3	Dune holds, post storm profile not safe enough
6	70	7.5	1:3	Dune breaches before the end of the simulation
7	100	7.5	1:2	Dune holds, gentler profile
8	100	7.5	1:4	Dune holds, erosion accelerated in early stages

Table 7. XBeach iteration runs and their parametrization.

At this point, having validated the XBeach model to be used (Appendix C), and with the settings and boundary conditions explained through Chapters 3 and 4 and summarized in Section 4.1, it is possible to run the XBeach model to assess the performance of the dune system, represented by the designed cross section, in the design 1/100 storm scenario. Depending on the results obtained, there are two possible outcomes: the design does not meet the requirements and needs more refinement going through another design iteration or the results are satisfactory and the current design is taken as the final one. In any case, it is still useful to go through different iteration processes to understand how the dune behaves and based on it estimate which design features can be modified to try to achieve a more efficient design, as it was done in this design process (Table 7).

The XBeach initial set up and the final output after the simulation of the design hurricane are shown graphically in Figure 38. This way it is possible to obtain a first insight of the dune ability to withstand the design storm boundary conditions. There is certain erosion taking place at the dune, leaving a reduced size barrier at the end of the event.

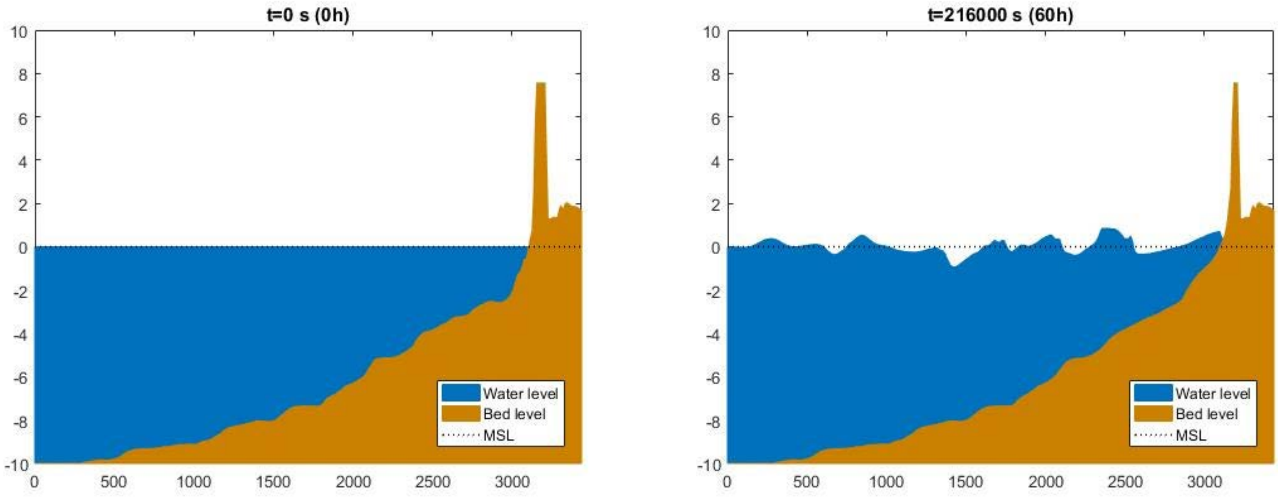

Figure 38. XBeach simulation beginning (t=0 h, left) and end (t=60 h, right).

However, as it was expected by looking at the design hurricane water level evolution (Figure 32), the dune profile development over the storm duration is not uniform. Erosion of the dune front takes place in different heights at different rates. Nevertheless, the trends can be categorized into four different stages, which correspond to the offshore storm surge development (Figure 39):

1. Wave impact at MSL
2. Forerunner
3. Surge peak
4. Surge lowering

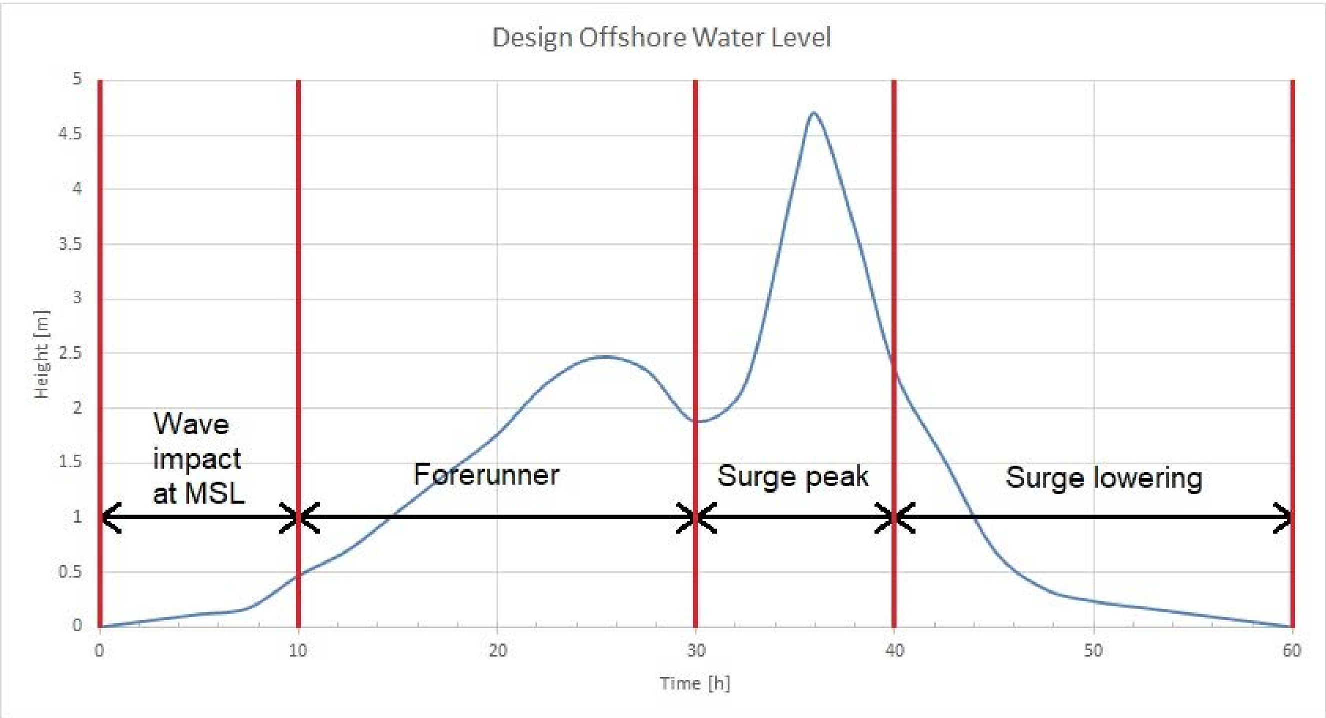

Figure 39. Design water level division into stages attending to dune profile development.

An analysis of these stages, as well as assessing each of their relative importance in the dune recession process will be done next, followed by a final reflection and conclusion of whether this design meets the requirements and expectative for being considered definitive.

4.4.1. Wave impact at MSL

The first storm stage comprises the first 10 hours. In this part almost no water level raise has taken place yet and the wave action is the main forcing. However, also the waves are still in relatively low height values (Figure 35) so the erosion is not significant. In fact the erosion is reduced further due to the vegetation effect, which plays a more important role in preventing erosion by dissipating wave energy when the waves are relatively low (Feagin et al., 2019).

The dune profiles at the beginning of the simulation and after 10 hours are shown in Figure 40. The only morphologic noticeable change is the recession of the shoreline and erosion of the beach before the dune and the deposition of this eroded sediment further offshore in the shoreface.

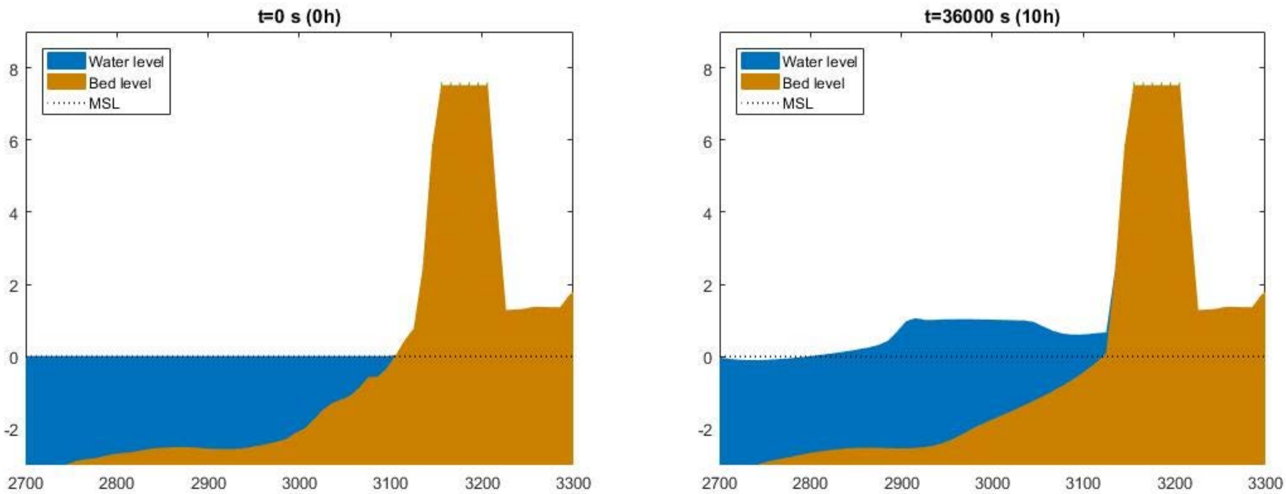

Figure 40. XBeach dune profiles at t=0 h (left) and t =10 h (right).

4.4.2. Forerunner

Although it is not explicit in the stage name, the wave action is still present during all the storm simulation stages. Furthermore, their height is increased compared to the previous time (Figure 35). However, their effect is bound to the water level at which their impact takes place, making the surge level a more important factor than the wave height itself.

The time when the forerunner of the design storm is taking place is between 10 and 30 hours. During this period the water level steadily raises to a relatively high value compared to previous situation (MSL +2.5 m), it is briefly maintained and lastly it experiences a small drop (Figure 32). The beginning and end dune for this stage is graphically presented in Figure 41.

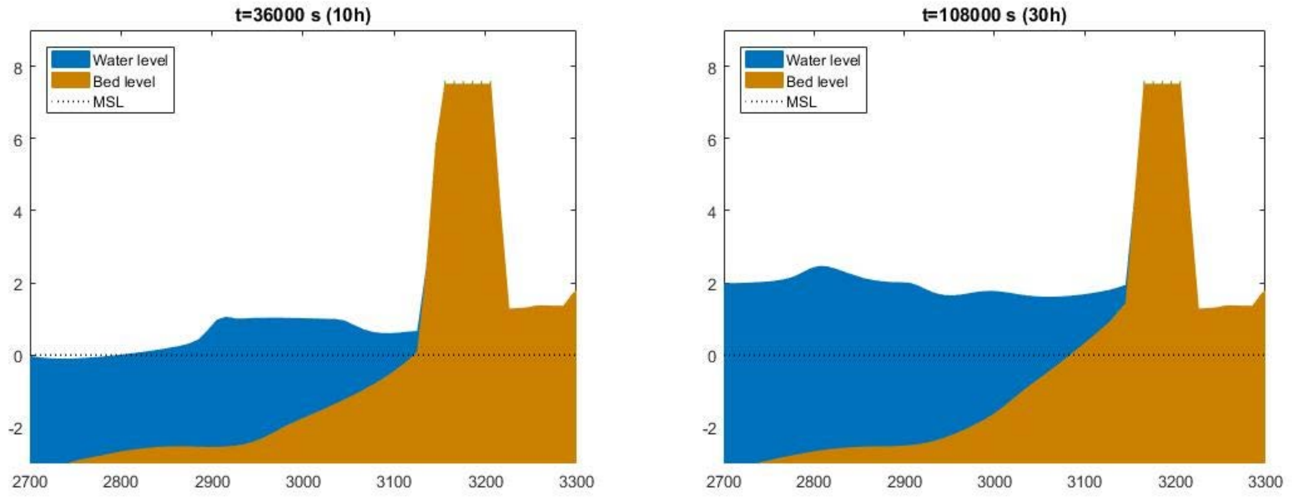

Figure 41. XBeach dune profiles at t=10 h (left) and t =30 h (right).

In this case a clear change in the cross section profile can be seen. With the water level increase, the erosion takes place higher in the dune front, leading to a collapsing effect of the upper parts and accretion at the foreshore and shoreface of the coast. This process leads to the beginning of the dune crest width reduction (ending in the next stage), which can result in ultimate failure if the forerunner extends abnormally in time. However, as explained previously, the design forerunner duration has been taken with enough safety margin to design for an unfavourable case.

In this part it is important that the dune is robust and wide enough, since the erosion here is prolonged in time. An inadequately dune without sufficient width would not be able to last for the whole storm duration, being the experienced retreat enough to completely wash it away or, not being sufficiently prepared for the surge peak.

4.4.3. Surge peak

This part is the one that takes the shortest time, but it poses the greater risk since failure of the dune by this stage would suppose a rapid inflow of huge volumes of water into the protected area. In this case it is considered to be located between 30 and 40 hours into the storm, but the absolute peak of the storm (4.7 m) lasts for 1 or 2 hours around the 36th storm hour.

Overtopping can be fatal for this design, since it can overwash completely the top of the dune if the discharges are too high (Steetzel & Visser, 1992), leading to a chain reaction where the lower parts as well as the contiguous ones are vanished as well. In order to prevent this situation, the dune should have resisted the forerunner forcing and the dune should be ‘healthy’ enough to withstand this last critical hydrodynamic forces.

The performance of this design when the peak water level is acting on the dune (Figure 42), even though there is a noticeable change in the overall dune width in a little time span compared to the previous ones, it still withstands after the greatest hazard of the whole storm. This faster erosion rate can be explained based on two factors, the higher water level makes the erosion take place in a greater portion of the dune and this is the time where the waves have the highest significant wave height. Furthermore, the water depth has increased in front of the dune, attenuating the depth induced wave breaking beneficial effect for load reduction.

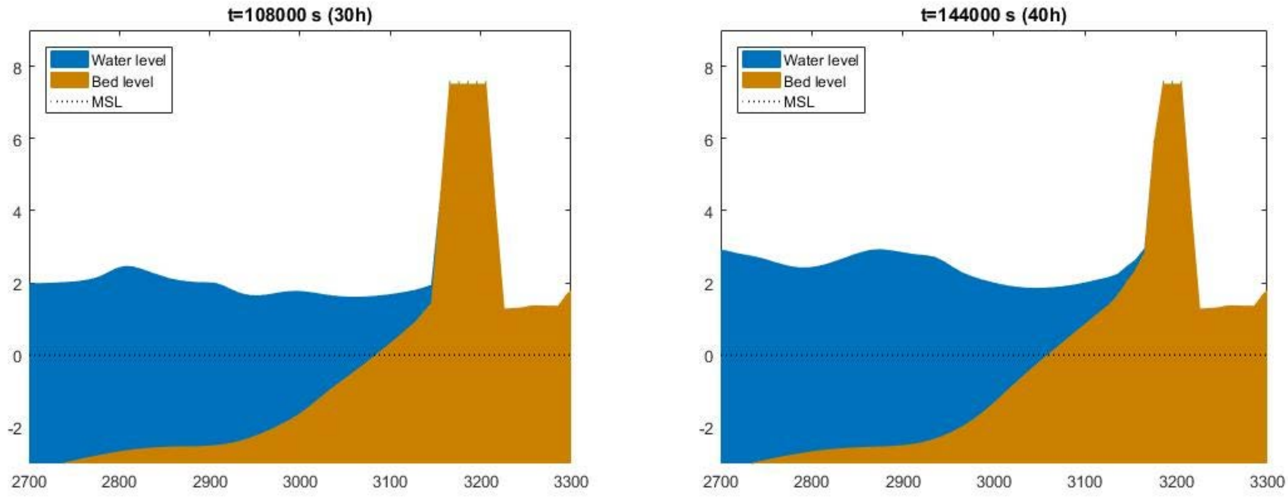

Figure 42. XBeach dune profiles at t=30 h (left) and t =40 h (right).

Similarly of how dune width was crucial when holding back the forerunner to plan against excessive dune recession and erosion, during the surge peak the crucial parameter is the height of the dune. It would not matter if the dune is extremely wide if it is overtopped in excess, the erosion would escalate exponentially until ultimate failure occurs. Thus the two main features initially estimated in Chapter 3, height and width, are indeed playing a crucial role when it comes to designing a dune capable of acting as a flood defence structure.

4.4.4. Surge lowering

The last hurricane stage is the process of coming back to normal water level and wave conditions, starting at 40 hours until the end of the storm at 60 hours. In Figure 43 it can be seen how the top part of the dune is not affected anymore by the hydrodynamics due to the low water level and reduced significant wave height.

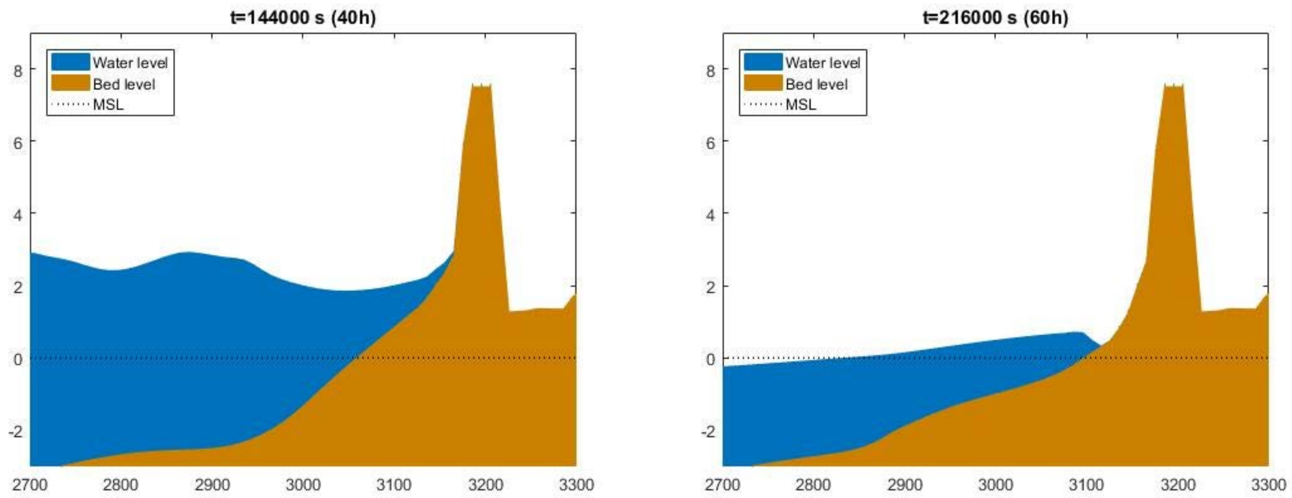

Figure 43. XBeach dune profiles at t=40 h (left) and t =60 h (right).

Most of this storm stage affects the lower part of the dune and coast shoreface. The process consists of the erosion of the dune toe and deposition further in the shoreface. This is due to the episodic event that just took place, which greatly disturbed the equilibrium profile the beach was heading towards. A storm erosion profile was briefly stablished which differs significantly from its regular one. Thus, in the next hours after the event, the morphodynamic effects are the cause of a redistribution of the sediments to a more dynamically balanced situation.

The process of a beach reaching its equilibrium profile can take years (if ever reached) (Stive & de Vriend, 1994), however, due to the storm the profile changed so much that when normal conditions are met again the first changes of the profile towards a more stable situation are visible within hours (List & Farris, 1999).

4.4.5. Discussion

The designed dune cross section has resisted the design storm conditions, however, further reasoning has to be made in order to qualify it as a reliable mean of flood defence. A failure in the system can result in breaching all over a great stretch of the coast, which would have severe consequences in terms of flood damages, ecology losses and even life loss. Thus extra security has to be given in the design of such a structure that suffers from high vulnerability and sensitivity after its ultimate limit state has been reached and failure has occurred.

In line with the mentioned safe designing, the model has been performed with worst scenario design parameters. Examples of those are the extraordinarily long and high forerunner, the first overtopping design being done with the absolute largest significant wave height in the series and the extensive duration of the storm when obtaining the first design dune width via storm dune recession.

In order to have a visual tool to make a judgement whether the dune design is safe enough or not, Figure 44 presents the situations before and after the storm overlaid, which allows to see the total dune erosion that takes place over the whole process and where the respective accretion is located. The red area is the sediment erosion, present mainly in the dune front area, which takes place during the storm and is redistributed towards the nearshore zone, with the light green area representing the accretion. The total amount of erosion due to the design storm conditions is 187.5 m^3/m. It can be seen that almost the entirety of the sediment exchange during the storm takes place along the first 500 m of coast, Figure 45 focuses in this active coastal zone.

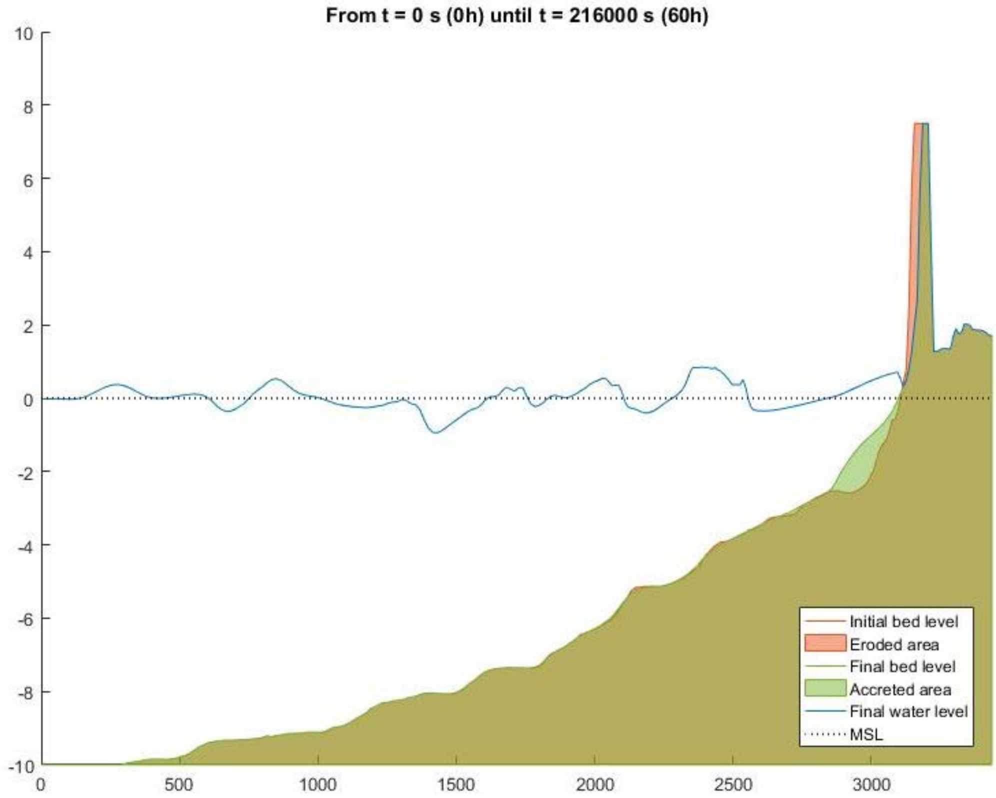

Figure 44. Xbeach design cross section bed level before and after the storm.

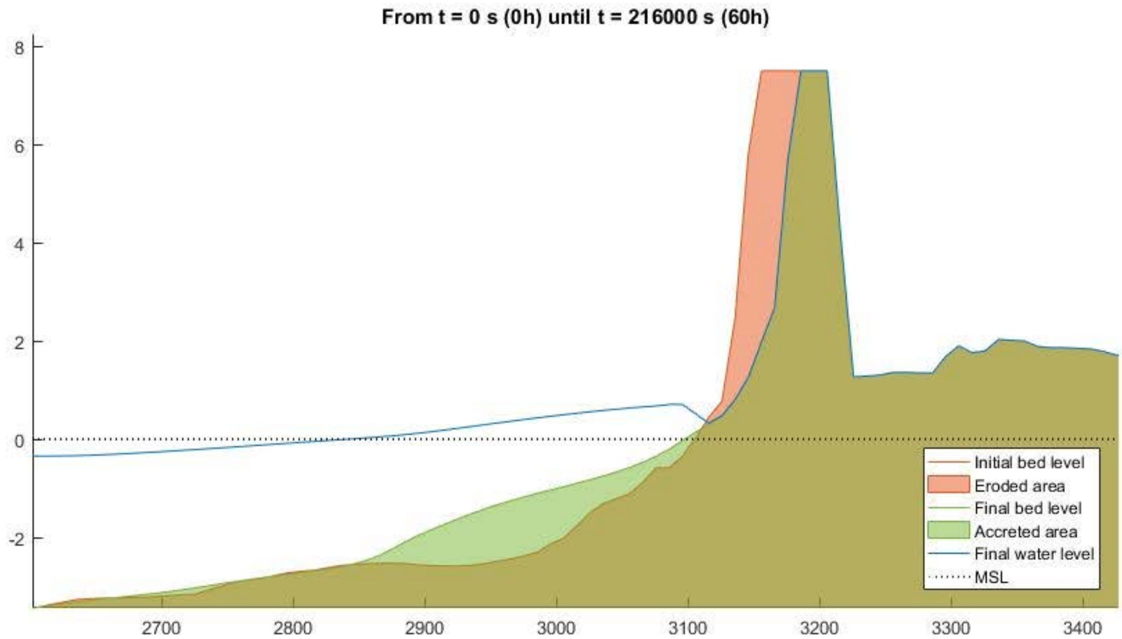

Figure 45. Xbeach design cross section active coastal zone bed level before and after the storm.

Crest width reduction can be taken as an estimate of the dune residual safety after the event. As long as crest height remains in its design level, overtopping is under control, preventing more hazardous situations for the dune safety such as overflow. In this case from the initial 55 m crest width, 35 m are eroded away, leaving the final dune with 20 m remaining. This translates into around 40% of the crest remaining. This value provides enough confidence in terms of safety standards to withstand the design 1/100 conditions.

Furthermore, it may look like that the design is excessively dimensioned and that it can either be lowered, or at least its width can be decreased. However it is necessary to broaden the protection perspective to the whole extend of the system. So far the model has been running one-dimensionally a single cross section although the dune whole system should be running for around 80 km long, which decreases the reliability of the design due to the variability of the different locations.

Throughout this section the design dune has been tested in the design location profile using XBeach numerical model and it has been determined that the design complies with the established requirements, meeting the desired safety standards of not allowing excessive overtopping that would potentially lead to dune collapse and hinterland flooding.

4.5. Further Optimization Options

The dune system is designed to fulfil its function of providing flood safety in Galveston Bay area without leaving unprotected any area along its coast. Through modelling it has been tested in a one dimension design scenario and it has performed successfully. However, there are two extra design features worth developing in more detail: dune shape and vegetation.

4.5.1. Dune shape

Looking back in detail at the XBeach simulation development, specifically during the forerunner and surge peak stages, a design correlation rule can be established: dune width determines the duration of the storm the dune can withstand without being excessively eroded and the dune height dictates the intensity of the storm that can be resisted without experiencing critical overtopping that would start erosion of the crest, ultimately leading to the failure of the system.

This double relationship (dune width with duration and dune height with storm intensity or peak) could be used beneficially for the design. If a wide enough dune resists the duration and high enough dune holds back overtopping, it is reasonable to think that the top part of the dune does not require to be excessively wide. Changing the shape of the design into a ‘berm-like’ dune in the front followed by a taller and more slender crest behind could theoretically fulfil the flood prevention requirements while optimising the amount of sediment requirement for its execution.

A design following the explained two heights concept has been modelled. The initial configuration is displayed in Figure 46. Out of the initial 55 m of crest width, the front half of it was lowered to MSL +3.2 m from MSL +7.5 m. The value height at which the berm is located has been chosen to be higher than the design forerunner surge (MSL +2.5 m) so overwash does not occur over that prolonged period of time. The graphical results obtained after the 60 hour simulation time of the design hurricane is presented in Figure 47.

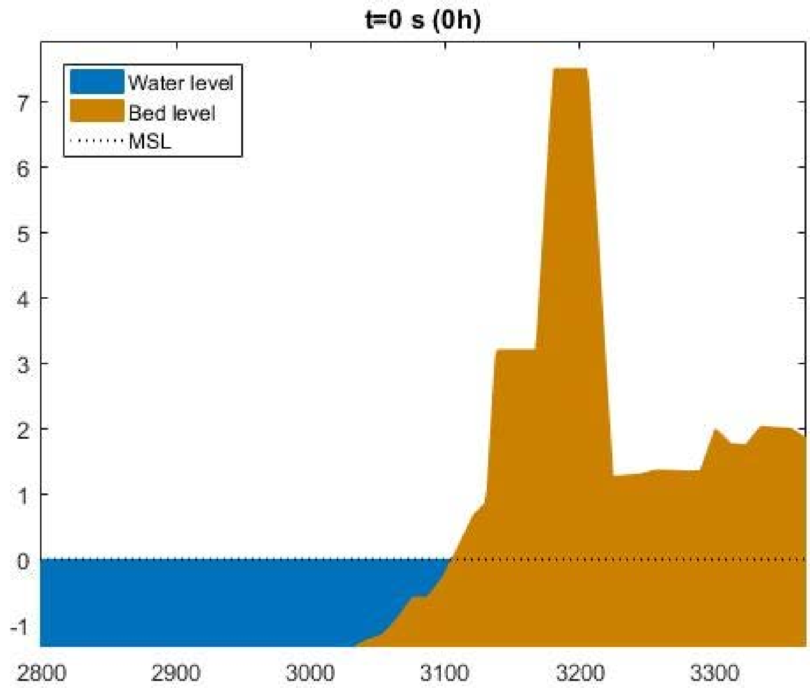

Figure 46. Xbeach dune design with a sandy berm initial set-up.

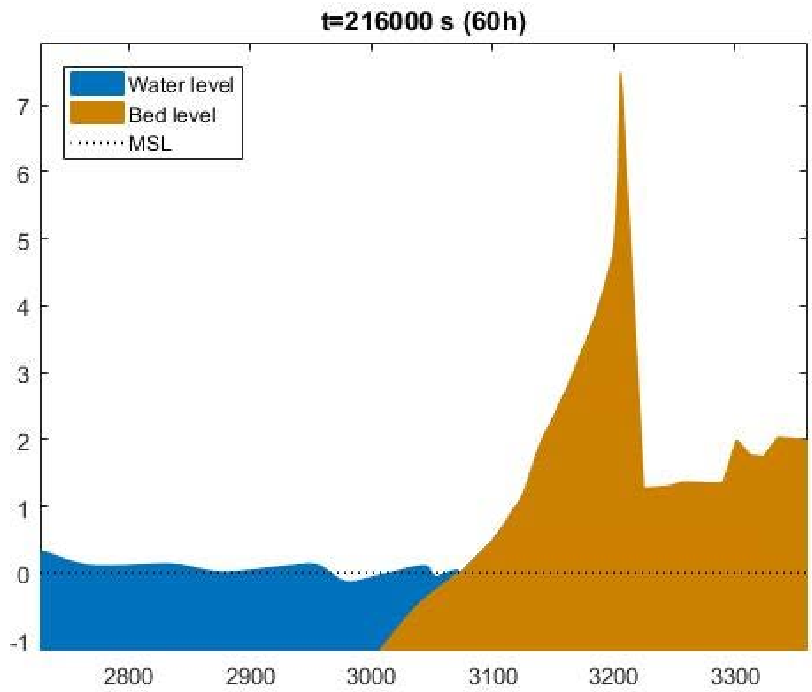

Figure 47. Xbeach dune design with a sandy berm at the end of the run.

Definitive conclusions can be drawn from the run. Although the dune didn't collapse for the entirety of the run, the crest barely survived as most of it was washed away. During the simulation the erosion rate of whole dune was significantly higher than in the previous section design. The berm did not last the whole forerunner due to the lack of sediment on top of it, which in the previous case was giving robustness to the design. This made almost half of the dune being eroded away before the surge peak takes place, a really inefficient use of the material since that is the critical part.

The only way that this concept idea would have been considered to be implemented in the final design is if the results yielded relatively similar results as in the previous section design since safety margin is an important factor in such a global design as explained beforehand. With this negative conclusive results the option of further research the optimisation of the dune shape with this approach is discarded.

4.5.2. Vegetation effect

In the design model (Section 4.4) vegetation has been implemented following the parameters used in the XBeach validation (Appendix C) that give the most accurate results in terms of dune storm behaviour. The vegetation that serves as basis for modelling is taking from the local ecosystem of Galveston Bay dunes (Howard et al., 2013) There are two types that can be found specifically in the dunes: marsh hay cordgrass (Spartina patens) and Bitter panicum (Panicum amarum) (Figure 48). The first one is usually found in the foredune area while the second one is present along the whole dune and its presence helps in greater amount to prevent erosion due to its characteristics.

Figure 48. Marsh hay cordgrass (Spartina patens) (left) and Bitter panicum (Panicum amarum) (right).

Based on this local vegetation, characteristics as similar as possible were used as vegetation input in the model selected based on previous studies in the matter (Vuik, Borsje, Tomohiro, & Jonkman, 2016). The used vegetation properties are presented in Table 8. The location of the vegetation starts at the bottom of the dune MSL +1 m, it covers its front, crest and the top part of the back (until MSL +4.5 m). Since the vegetation in the local dunes is sparse and not uniformly distributed, a relatively low vegetation density value was expected, as Table 8 shows.

Properties	Value	Units
Height	0.25	m
Stem diameter / Blade width	0.01	m
Density	500	units/m^2
Drag coefficient	0.4	-

Table 8. Model vegetation characteristics.

The influence of vegetation in the erosion process during the storm is uncertain, thus a simple sensitivity analysis can be performed. The expected erosion mitigation of the vegetation in the process is relatively low, since these type of plants are not big in comparison with the forcing. However, vegetation produces short wave dissipation at a certain degree.

The final model erosion after the storm is compared with the same situation without the vegetation implemented in order to assess the validity of its inclusion. Figure 49 shows both situations. As stated in previous sections, the total erosion in the design case with vegetation is 187.5 m^3/m, while if the vegetation is not present, the erosion is increased to a value of 225.5 m^3/m. The difference is an increased erosion of 20.3 % in the case without vegetation with respect to the one including it.

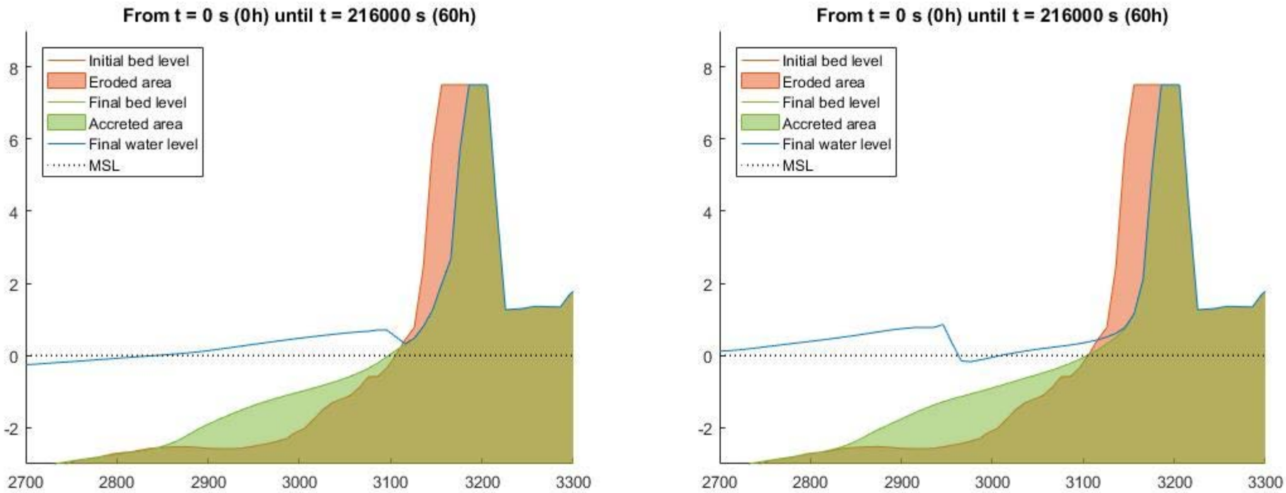

Figure 49. XBeach dune simulation results with (left) and without (right) vegetation.

The top part of the dune and the crest show no differences across both scenarios, the surge peak is fast and has a strong enough impact to be even slightly mitigated by the vegetation. However, at the dune of the foot (bottom 2 m), there is a difference in the way they are shaped. Since most of the low energy forcing is acting at low storm water heights and their respective associated waves are also not extreme, the dissipation here is noticeable when looking at the profile. The vegetated dune is less eroded at the toe, preventing some of the sediment collapsing. A well-developed and maintained root system is mandatory for these favourable features to take place, otherwise the plants would just not withstand the wave effect and would be uprooted unable to provide a time lasting passive action.

The model shows that the vegetation indeed mitigates the storm ongoing erosion process, mainly at the toe of the dune. This fact is beneficial for a post-storm dune recovery scenario because, although it does not look like a big difference, when extending the design along the whole Galveston Bay coast the total prevented eroded volume can make a difference when it comes to dredging the sediment back to the dune for its reconstruction.

Furthermore, the limited vegetation influence provides reliability into the design since it is not one of the strongest measures to mitigate hurricane effects. It is good to include it in the model and design plans, but in this case its hydraulic functions are secondary yet beneficial. On the other hand, the use of this local plants is largely influential in the integration of the land barrier with its environment, contributing to the landscape and ecosystem.

5. Costs

5.1. Construction Costs

One of the main factors determining the feasibility of a design that fulfils the required necessities with respect to other alternatives is the economic aspect. When considering a design plan of this magnitude, the construction or upfront costs are a major limiting factor due to the necessity of an efficient use of the available budget. The total direct costs are to be estimated from the sediment volume needed and based on them, total construction costs will be obtained and compared to those of the Coastal Spine land barrier proposal.

Project direct costs are proportional to the construction material needed. In this case since the planned dune consists exclusively of sand, the total volume of sand needed determines the direct costs. In The Netherlands the unit cost per m^3 of sand is relatively low, usually ranging between 3 and 5 €/m^3, occasionally reaching a price of 10 €/m^3 (Schasfoort & Janssen, 2013). However, quality sandy sediment is scarcer in Upper Texas Coast since it is not easily reach by current dredging methods. An initial unit cost for sand meets the standards for this project emulating the one found in along Galveston coast would be between 45 and 50 \$/m^3 (United States Army Corps of Engineers (USACE), 2018). A value of 50 \$/m^3 is taken.

Some of the most feasible nearshore locations to extract sand for the project are Rollover Pass, the area of Bolivar Roads, San Luis Pass or the Brazos River delta further to the southwest from Galveston Island end. There are offshore sand resources potentially on the continental shelf available that, although harder to reach due to its location more seaward, hold a greater volume of adequate quality sand. Some examples are the old river valleys, Sabine and Heald sand banks and tidal delta deposits. Sand available onshore is discarded, its sand potential is substantially insufficient for a project of these dimensions making this source negligible (Anderson, 2007) (Frinkl, Andrews, & Benedet, 2004).

In order to obtain the total volume of sand needed to be placed during the construction phase, firstly the coast was divided into different sections, with different beach retreat rate between them (Paine et al., 2014). Cross sections with similar beach erosion over recent time are grouped into a stretch and the process is repeated for the whole coast. The stretches, their length and their representative retreat rate are shown in Figure 50 and Figure 51.

Figure 50. Coast section classification with similar retreat rate for Bolivar Peninsula

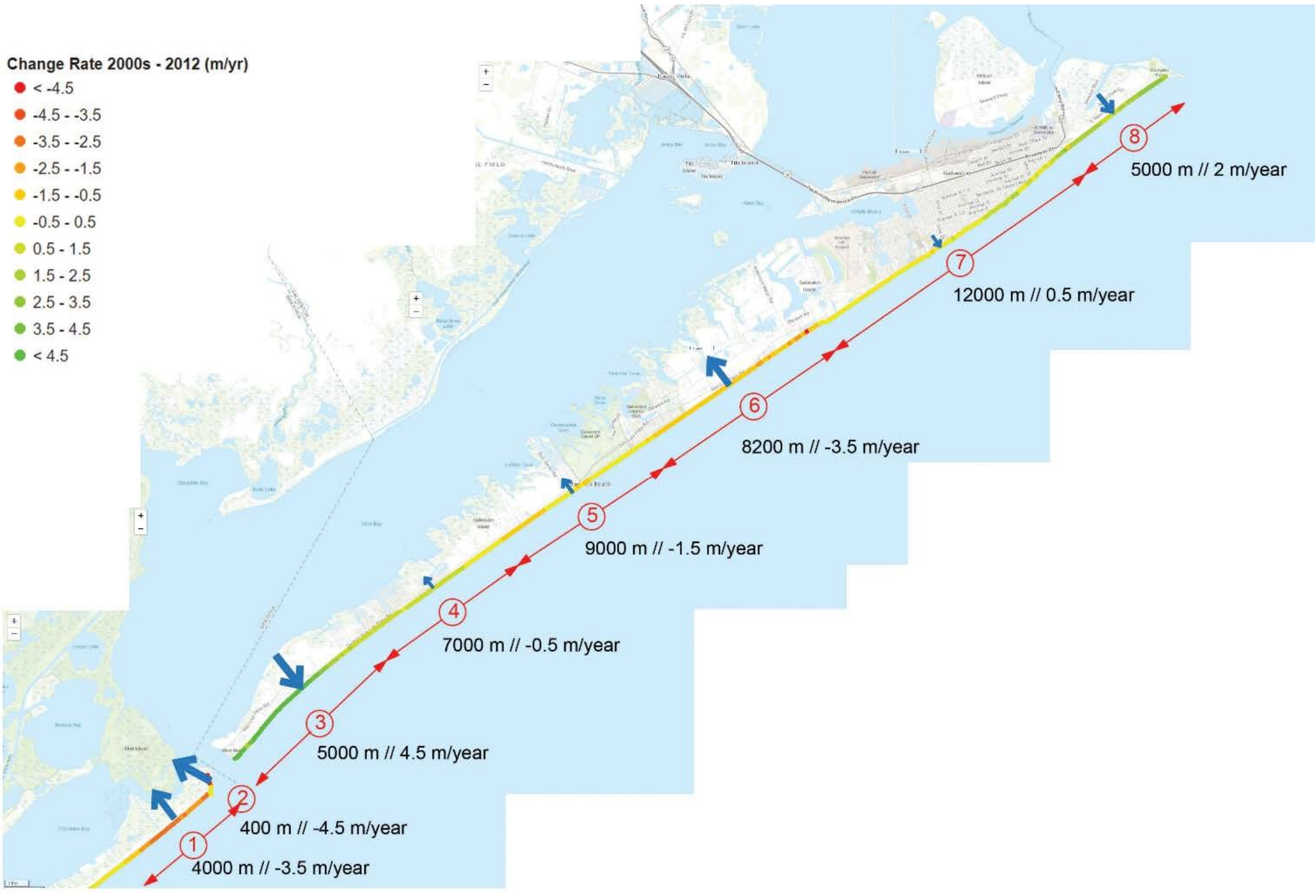

Figure 51. Coast section classification with similar retreat rate for Galveston Island

Transects with similar retreat are subject to comparable forcing, theoretically making their cross shore topography being shaped in a similar pattern. In each of these sections the dune is manually placed in its most fitting place, making use of the already natural dune to minimize the extra dune area thus optimising sand volume required and its associated cost. In Section 6.1 the location of the dune within the profile will be further discussed. The representative profile in each of the sections with the dune already projected in them can be seen in Figure 53 and Figure 52. The shades show the extra sand area needed for the dune placement, their individual values being also displayed at the right of each profile.

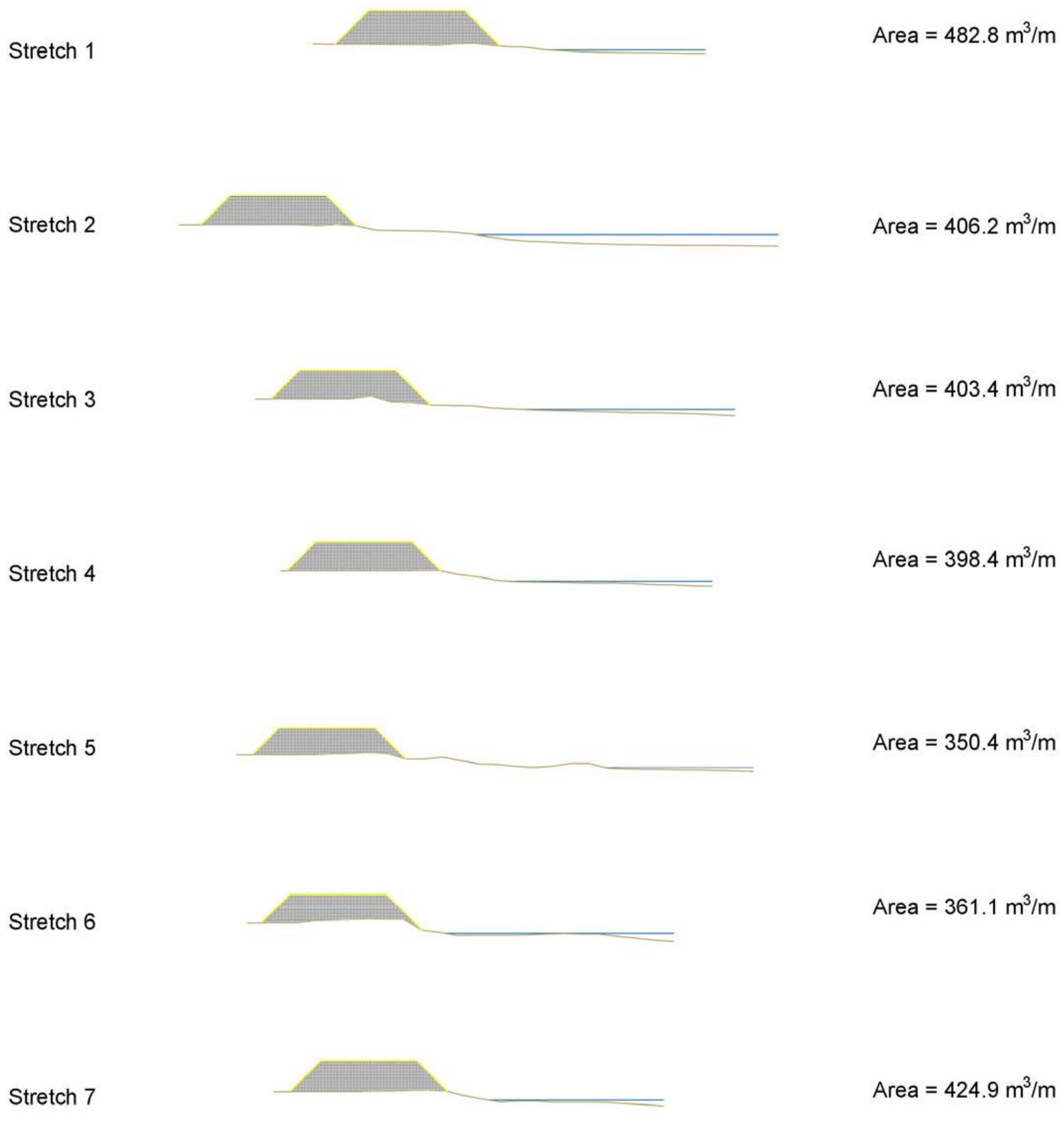

Figure 52. Planned coast profile of the representative cross sections of Bolivar Peninsula.

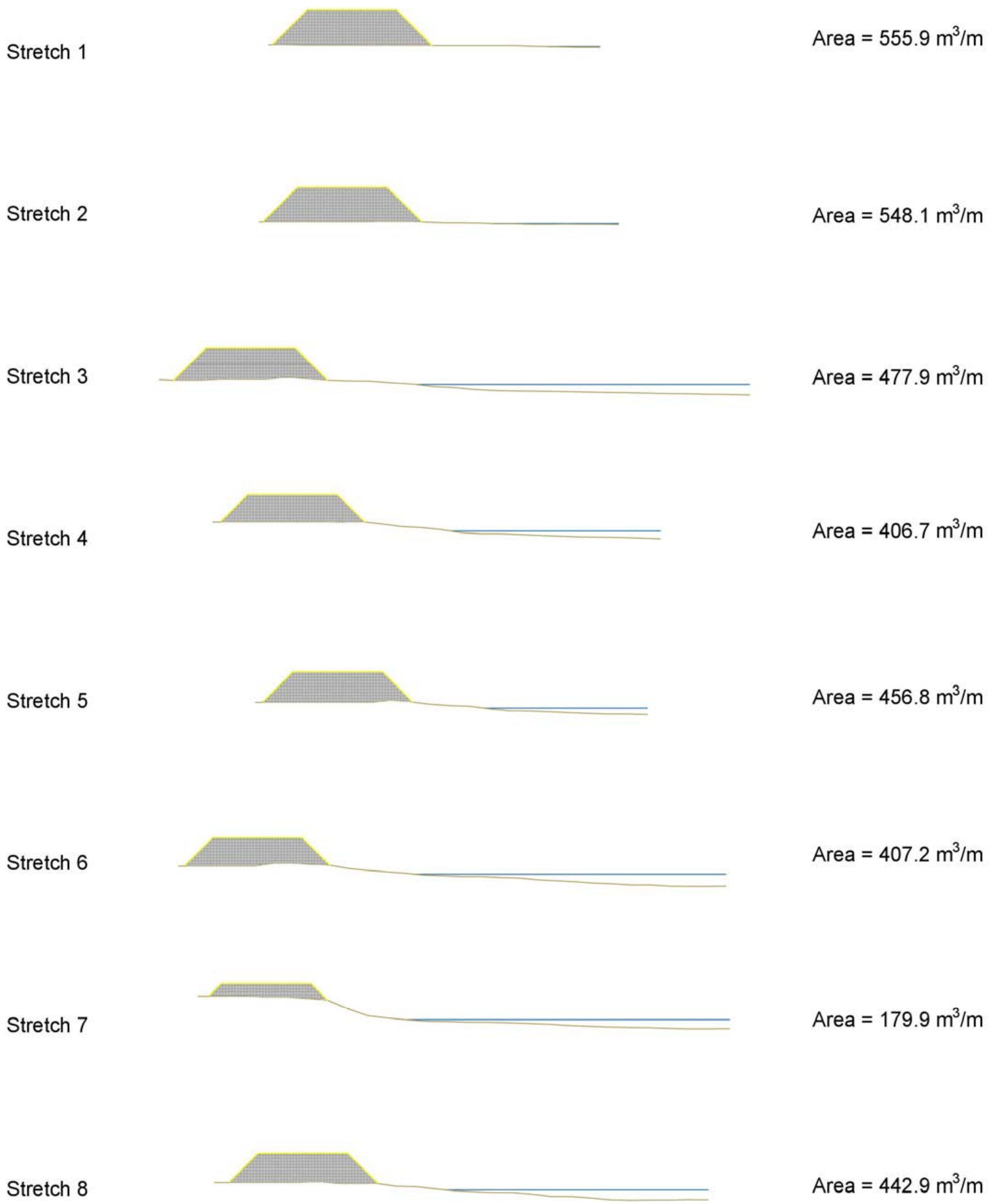

Figure 53. Planned coast profile of the representative cross sections of Galveston Island.

With the area of the dune in each cross section and the length of their respective stretch it is possible to obtain the total volume of sand needed for the dune system placement: 31.5 million m^3 of sand. With the established 50 $/$m^3$ sand unit price, the material construction cost of the project is **1,576,115,000 $**. Following *E.C. van Berchum et al., 2016* standard values and process of obtaining the total construction costs, a value of **3,061,110,900 $**.is obtained (maintenance and land buying costs are not included in this estimation, they will be addresses in subsequent sections) . The results from the cost estimation steps are summarized in Table 9.

Specification	Quantity	Unit price	Total
Sand	31,522,300 m^3	$ 50	$ 1,576,115,000
Incomplete design	25 %	$ 1,576,115,000	$ 394,028,800
Total direct construction costs			**$ 1,970,143,800**
Construction site costs (barrier location)	10 %	$ 1,970,143,800	$ 197,014,400
Overhead - Directional costs contractor	8 %	$ 2,167,158,100	$ 173,372,700
Profit	5 %	$ 2,167,158,100	$ 108,357,900
Total indirect costs			**$ 478,744,900**
Total estimated costs			**$ 2,448,888,700**
Unforeseen project risks	25 %	**$ 2,448,888,700**	$ 612,222,200
Total construction costs			**$ 3,061,110,900**

Table 9. Total construction costs estimation.

The default Coastal Spine land barrier dike design has an estimated total construction costs of $ 2,247,992,500 which is 26.6% cheaper than the total costs obtained for this thesis solution (Table 9). However, in *E.C. van Berchum et al., 2016* it is also assessed the possibility of a coastal dike, which total construction costs have been estimated as $ 3,465,209,000; a value 11.3% higher than the one obtained for this case in Table 9. However, the former less expensive alternative located inland was the preferred one against the coastal alternative in *E.C. van Berchum et al., 2016*.

The differences in cost can be explained via the design principle behind each design. While in *E.C. van Berchum et al., 2016* the structure is expected to withstand the same 1/100 design flood event, major damage to the structure is prevented using a top armour layer. This revetment is not present in this thesis design, reducing its implementation upfront costs but leaving it vulnerable to erosion in case of hurricane, which increases the maintenance costs in a post event scenario due to the need of partial reconstruction. On the other hand, long term maintenance in this dune design is simpler and more reliable to estimate than in the case of a traditional dike like the one in *E.C. van Berchum et al., 2016.* Furthermore, the flexibility of a dune system allows to tackle the sea level rise threat in a dynamic and progressively way. These recovery, maintenance and sea level rise design features, which are not analysed in depth in previous Coastal Spine reports, are developed in the next sections.

5.2. Post-Storm Dune Recovery

The purely sand based philosophy of this flood defence system carries with it the fact that, due to the lack of revetment or armour layer at the structure outer part, during an storm event, erosion takes place. This erosion, as seen in Chapter 4, can occur at different rates depending on the waves and, above all, storm surge. The higher the water level, the more surface is in contact with the water under the dynamic hydraulic forces and the more erosion takes place. This erosion process is the physical process working against the defence mechanism of the dune. The eroding ability of the dune can be seen as its resistance in the sense that, the more erosion it can hold, the longer the storm can be without failure or breaching.

Being this property as essential as it is, it is reasonable to plan for the recovery of the dune design state after its front part has been affected to keep the defence structure ready for when another event takes place. In Chapter 4 it has been modelled the change of the profile given the design boundary conditions as input. The final design dune profile at the beginning and end of the simulation (Figure 54) can be analysed in order to estimate the required sand recovery after a hurricane. The differences between these two profiles indicate the eroded and accreted area.

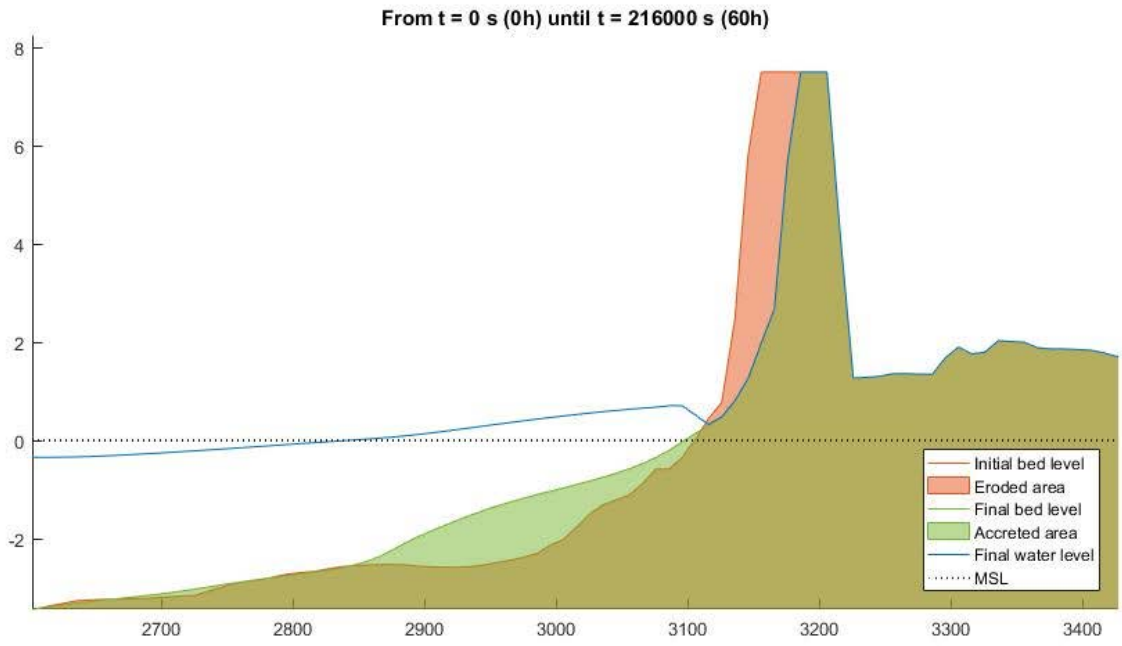

Figure 54. Xbeach modelled dune profile before and after the design hurricane.

Figure 54 leads to an early positive conclusion can be made, most of the eroded sediment from the dune front is deposited in the active coastal zone, relatively near to the coast compared with regular condition cross shore sediment transport, where the estimated Galveston Island depth of closure is 5 m (Wallace et al., 2010). This, together with the fact that almost no sediment is lost from the system, makes recovery actions easier and cheaper to perform. Within the first 250 m of the coast, 94.5 % of the eroded sediment is deposited (177.3 m^3 out of the total 187.5 m^3 erosion volume).

The total accretion volume of the whole 81.1 km coast needed to be dredged and built back into the dune sums up to 143,790,300 m^3. A feasible plan to perform the recovery of the dune system after a potential design hurricane landing that would take one year would be the deployment of ten cutter suction dredgers CSD650-18 with spud carriage and wedge piece (Figure 55) working 8 hours per day 5 days per week (Damen, 2109). This configuration would require a budget of $ 215,953,700 (Pristine Waters, 2019). It is needed to add to this value the cost of the 5.5 % of the sand lost in the system that would be needed to be bought again at its original price of 50 $/$m^3$. The total cost of this sand volume, applying the same construction cost rates as in Table 9, is $ 80,333,800. Thus, one design storm recovery process adds up to a total cost of **$ 296,287,500**.

As it was mentioned before, the fact that most of the eroded sediment is not lost in the system and can be accessible again to reuse is of vital importance for the economic aspect of the reconstruction. In terms of volumes and price, 94.5 % of the sediment needed for the recovery is dredged and represents 72.9 % of the recovery total costs; while the 5.5 % of the sand that is lost and needs to be bought again comprises 27 % of it. The cost of the reconstruction would escalate considerately the higher the proportion of bought to dredged sand would be.

Figure 55. CSD650-18 cutter suction dredger. Source: (Damen, 2109)

Since the design life of the defence system is 100 years and the expected return period of the design hurricane is also 100 years, it is expected, on average, one design storm event during its lifetime. This means that the cost of one recovery should be included in the total costs of the project, raising the value calculated in the previous section to **$ 3,357,398,400** (*E.C. van Berchum et al., 2016* is 67.7% of this price). Thus recovery costs entail a total cost increase of 8.8 %, which in a project of these dimensions, absolute costs and influence, is significant.

5.3. Maintenance Due to Lesser Storms

In the previous sections the construction costs as well as the costs of dune recovery from a design hurricane impact have been estimated. However, there is still another a factor playing an important role in the economic feasibility assessment of the dune solution: maintenance costs due to minor storms. The dune system is degraded not only when the design scenario conditions are met, but also anytime when any water forcing reaches the dune. When the 1/100 hurricane conditions take place the impact is instantaneous and can lead to catastrophic consequences, so the dune must be kept in good conditions during its lifetime to ensure its expected performance during such an event.

Along its service life, the dune is expected to be exposed certain hydraulic conditions that, even though not being as harsh as the design ones, lead to certain amount to erosion. As it has been stated previously, the dune is ideally placed away from the effect of daily hydraulic conditions to prevent excessive erosion and alteration of the coastal system. So, erosion of the dune only takes place whenever the water reaches the foot of the dune. This condition is met when the combined effect of surge and wave run-up is enough to reach the bottom of the dune.

Data analysis is performed in order to evaluate when the total water level reaches the dune and obtain its associated maintenance costs over its lifetime. The boundary conditions are available via NOAA stations: Station 42035 for wave data and Station GNJT2 and Station GTOT2 for water level data (NOAA, 2008). Their location is shown in Figure 56. Those times when the dune foot level is surpassed by water are modelled in XBeach and after analysing the results a conclusion about this long term maintenance can be reached.

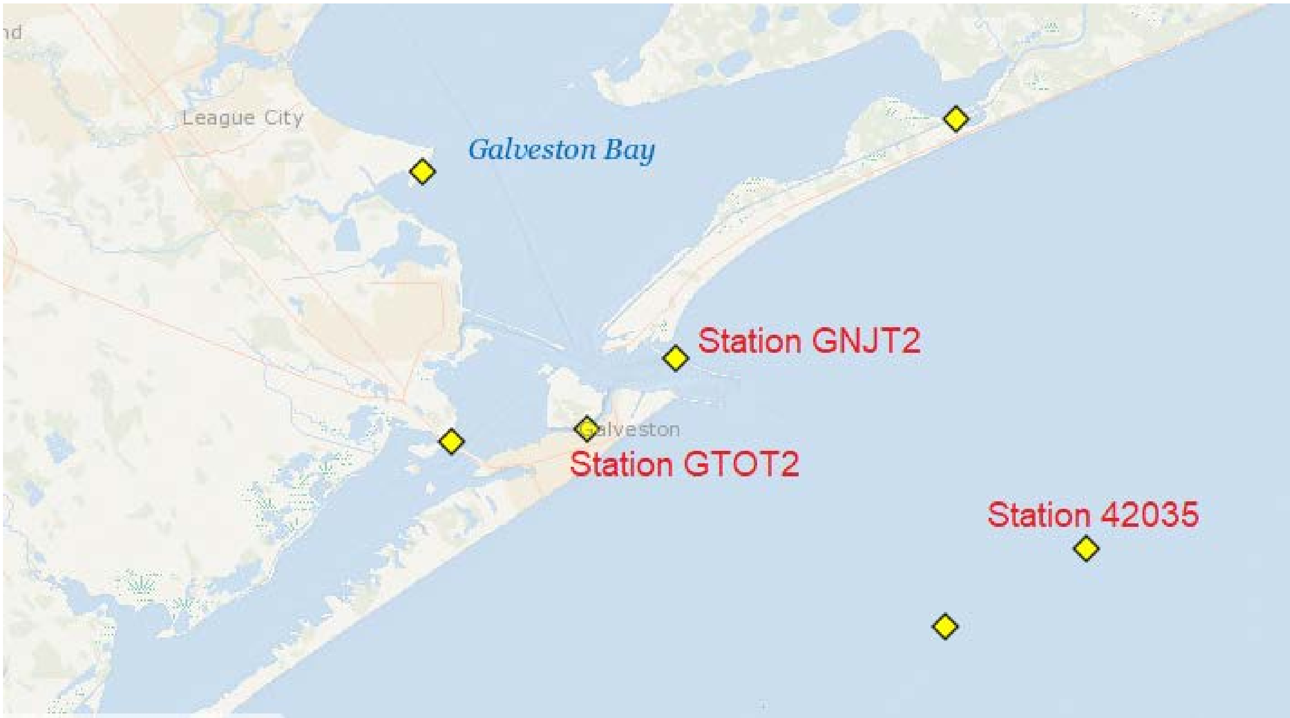

Figure 56. Wave and water level stations location. Source: (NOAA, 2008)

Station 42035 has been collecting wave data every hour since 1993 (Figure 57). The period between 1993 and 2017 accounts for 25 years, period of time which is representative for the storm magnitudes that reach dune bottom level. The whole analysis is thus performed over these 25 years of data and applied for the 100 year design life of the dune.

Hurricane Ike deserves a special treatment in this analysis. It took place in 2008, thus it is included in the data series. Nonetheless, it is an event with a return period way higher than the storms considered in this section, which can be considered as seasonal storms. It even is considered relatively close in terms of boundary conditions to the dune design storm. In this series, Ike is then an abnormal event that should not be taken as representative in this 25 year range of data analysis. Other events, such as Hurricane Claudette in 2003, landed in Galveston Bay in these 25 years, but this type of storms although impactful in the region, are more in line with the storm return periods expected within the range of years treated.

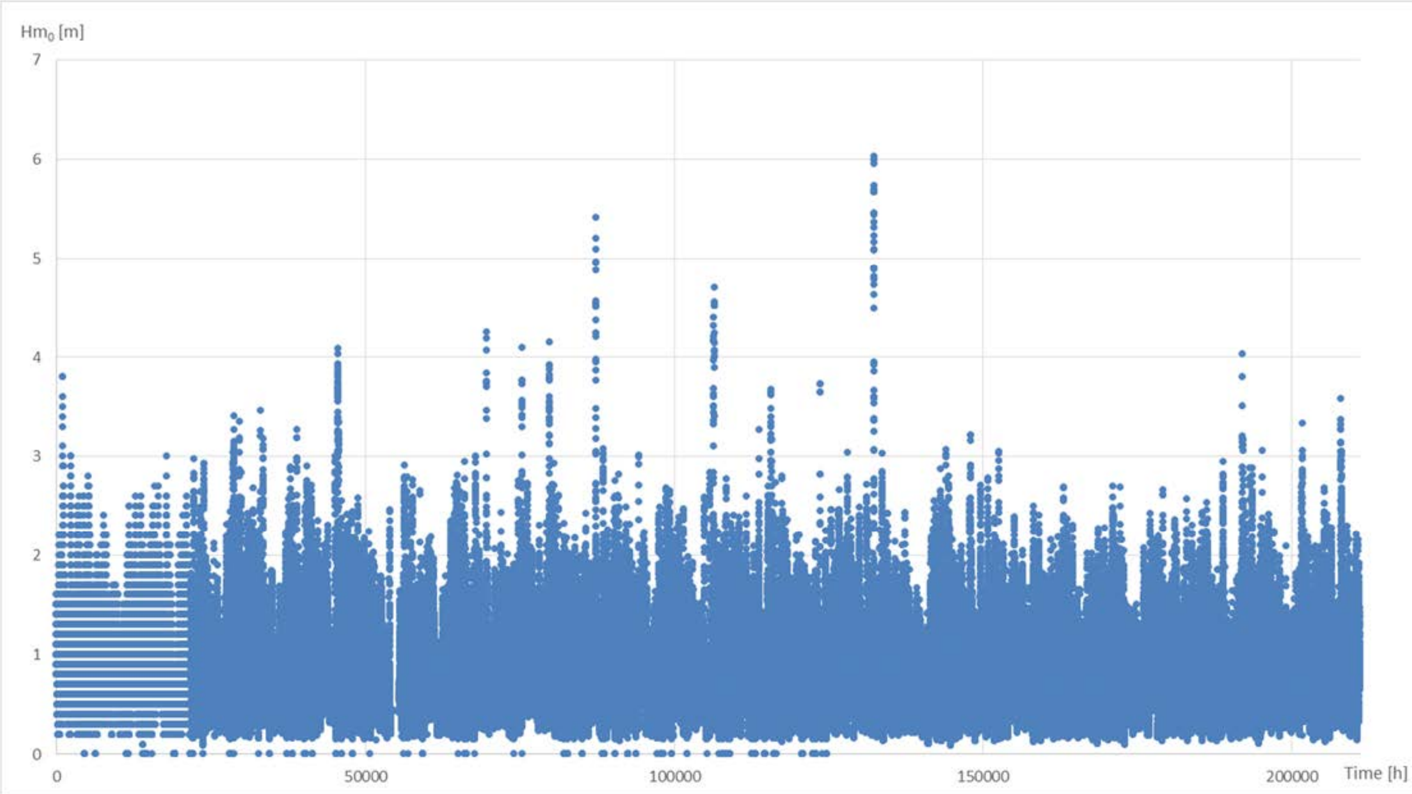

Figure 57. Station 42035 hourly significant wave height since 1993. Source: (NOAA, 2008)

For design purposes, in order to obtain a dune foot level, an average of the projected dune sections of Figure 52 and Figure 53, weighted over their length (Figure 50 and Figure 51), is calculated. The result is a design dune foot level of MSL +1.47 m.

The equation used to obtained the wave run-up is the following (Van der Meer et al., 2016):

$$\frac{R_{u2\%}}{H_{m0}} = 1.75 \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \frac{\tan\alpha}{\sqrt{\frac{H_{m0}}{1.56 \cdot {T_{m-1,0}}^2}}}$$

Equation 1. Run-up formula.

The beach slope is obtained the same way the dune bottom level was, averaging the sections over the length, obtaining a value of 1.78º. 7 s is the preliminary spectral wave period. In order to not underestimate the erosion the berm, friction and oblique wave attack factors are taken as 1 since none of those parameters have a significant presence in this beach configuration before reaching the dune.

The other factor determining whether the dune is eroded or not due to lesser storm conditions is the storm surge. By combining it with wave run-up, the total water level is obtained and if it exceeds the dune foot level, it is accounted for the Xbeach simulation. Station GNJT2 is in an ideal spot in order to obtain water levels resembling offshore conditions because it is located in the inner side of the jetty, where the Houston Shipping Channel is, an area which is considerably dredged, thus unaffected by nearshore coastal processes that may alter the recorded water level from those taking place in offshore waters. However, it only has data since 2001 and its functioning has repeated interruptions, substantially reducing its sample data.

To compensate this lack of data recorded in Station GNJT2, Station GTOT2 is used. Water level data from 1993 until 2017 can be obtained via this station, but its location is not ideal. It is situated in the inner side of the bay, in a channel like water mass between Galveston City and Pelican Island. This specific location does not capture water level in the same way as this case needs, from offshore toward the Gulf coast. Combining both stations is done in order to overcome this lack of data at the desired location.

Firstly, for Station GNJT2, the wave run-up obtained from the 25 year significant wave height hourly series is added to the corresponding water level taking place at the same time, leading to the total water level at each time. Then, the surges of those hours at which the total water level exceeds the dune bottom level are compared to the surges at those same times obtained from GTOT2. A relationship between surge levels at the jetty over surge levels in the inner bay is then obtained for the cases in which the total water level exceeds MSL +1.47 m. The average of these relationships is 1.05. This storm inner bay to coast surge conversion factor is applied throughout the GTOT2 surge data series to obtain the equivalent surge at the coast side.

By applying the coast over inner bay storm surge relationship it is possible to overcome the data scarcity at the station of interest by transforming the data from the other related station. Now, those cases in which the combination of wave run-up and (newly obtained) storm surge exceeds dune foot level are taken and run in the Xbeach model.

The results of the aforementioned data analysis methodology yield 708 hours in which the total water level exceeds MSL +1.47 m, with an average water level of MSL +1.66 m and maximum of MSL +2.7m. The mean wave spectral period during this hours is 7.6 s, which makes acceptable the assumption of 7 s taken earlier in the analysis.

The newly obtained series of waves and surges that combined surpass the dune foot can now be used as an input in XBeach in order to obtain the erosion caused by lesser storms. The erosion quantification can be obtained from the difference between the start and end profile after 708 hours of simulation Figure 58.

In contrast to what happened with the design storm, it can be seen in Figure 58 that in this case the longer although milder conditions causes the erosion of the surf zone and foreshore. This can be expected due to the lower waves and water level, which makes the waves break closer to the bottom thus eroding this part instead of the dune itself, where the water reaches without enough energy to transport sediment from it. The eroded sediment is deposited evenly across the shoreface, smoothing the profile. The offshore sand bar is also eroded, effect which resembles the winter profile development under more energetic conditions in the summer/winter seasonal changes (Larson & Kraus, 1994).

Almost the entirety of the erosion takes place nearshore, which is the place that needs to be maintained to keep a functional flood safety system. This beach area is of paramount importance because the decreased water depth in this area makes bigger waves break thus reducing the hydrodynamic load at the dune face, as well as being one of the main recreational features of the area.

The nearshore area erosion in the simulation is 120.36 m^3/m. As this simulation accounts for a period of 25 years, 4.81 m^3/m is the expected yearly erosion. The system length is 81.1 km, thus the erosion across the whole system is 390,447 m^3/y. Thus during its service life of 100 years, a total of 39,044,784 m^3 of sand erosion is expected. This value holds a certain amount of uncertainty due to the heterogeneities across the coast and dune, input assumptions, model inaccuracy and the stochastic nature of the storm hydraulic boundary conditions that may take place along the dune system lifetime.

In order to decrease the maintenance costs, those sediments that are located closer to the shore (in the first 250 m) can be reused at the dune. In this case, 35% of the nearshore eroded sand is deposited close enough to be dredged at a more economic rate than purchasing new sand. The total cost of dredging this material is estimated to be $ 20,524,000. The remaining 65% has to be acquiring again at a unit cost of 50 $/m^3, yielding a cost of $ 2,464,640,000 (applying the same rates as in Table 9). The total combined cost of the sand from both sources adds up to **$ 2,485,164,000**. As in the previous section, there is a vast economic difference between dredging accreted sediment and getting new one.

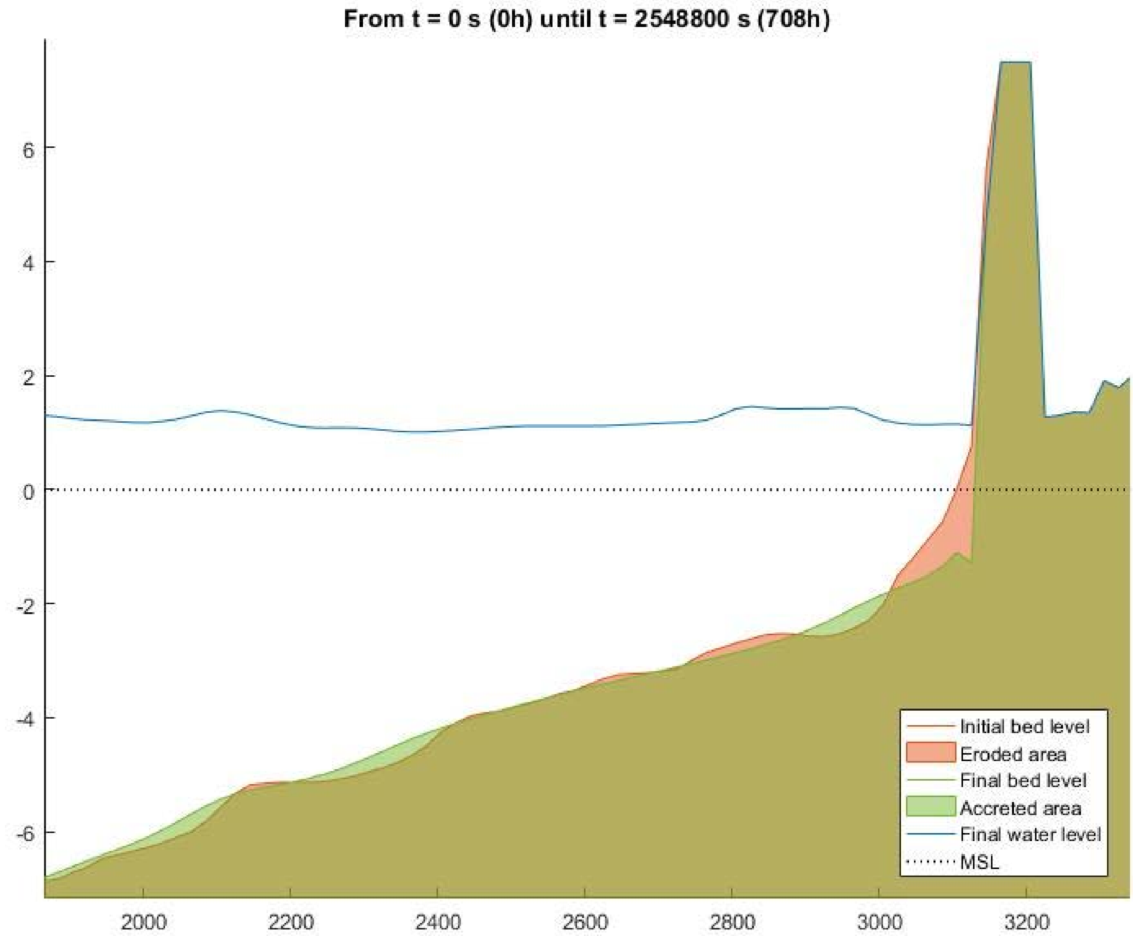

Figure 58. Xbeach profile at the start and end of the lesser storms simulation.

This maintenance cost is **18.8 %** less than the total construction costs, making them relatively similar quantities. In other words, with the analysis done it can be concluded that the yearly cost maintaining the dike and beach in design conditions due to lesser storms action would suppose less than 1 % of the total construction costs.

5.4. Sea Level Rise Dune Upgrade

The last feature regarding dune maintenance cost that makes this design different from the land barrier presented at *E.C. van Berchum et al., 2016* is the possibility of adapting the design progressively to overcome the sea level rise issue. When designing for a lifetime as long as 100 years, it is necessary to plan for likely boundary conditions future change during this time. One of these changes, which is certain to be developing over time, is sea level rise.

One of the strengths of the dune approach instead of a traditional dike is its flexibility. Due to its purely sandy composition, any change to its structure is relatively easy to perform, it is a uniform material that is not hard to place or move, so future modifications to the design would not pose complex execution problems. This flexibility can be used in order to tackle sea level rise over the dune design life.

Even though there is some degree of uncertainty in quantifying future sea level rise, over the next 100 years a height increase of 1 m is expected to take place at the Galveston Bay coast (de Vries, 2014), value that is used in this design. Figure 59 depicts how the basic dimensions of the model design dune would be altered by increasing its height 1 m while keeping the same crest width and slope angles. The extra volume needed to reach a dune height of MSL +8.5 m is 103 m^3/m. Since Figure 59 represents the dune without the topography, the specified volume needed to increase 1 m its height is the maximum one, since the integration of the dune within the cross section would reduce the amount of material needed in some places.

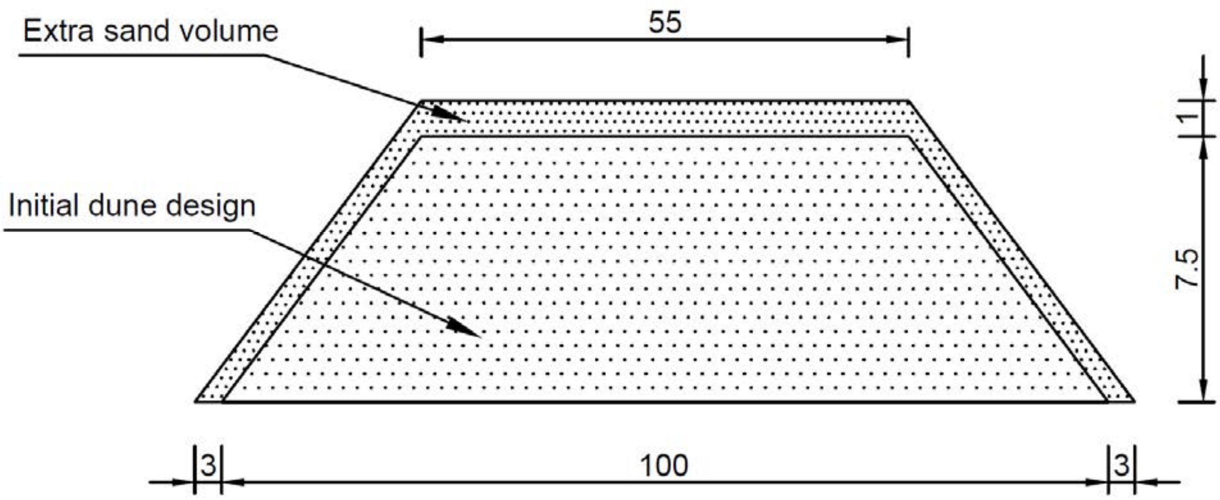

Figure 59. Dune design with extra sand volume to account for 1 m height increase (units in metres, vertical scale enhanced four times)

One of the aforementioned advantages of the sand solution is the possibility to adapt progressively to the ever changing conditions. In this case, 1 metre of sea level rise does not take place instantaneously, but over the course of 100 years. This means that the modification to the design can be done over time. An advantage of this approach is the possibility to overcome any deviation from the change of conditions prediction, for example if the increase in sea level is higher or lower than expected. Assuming the exposed parameters as valid, and that a certain dike heightening protects against the same amount of sea level rise, in this case the change to the profile can be approached as a required sand volume of 1.03 m^3/m/year over 100 years, which makes up for the total volume over the course of the dune design life. It does not mean that the volume has to be added to the dune on a yearly basin, but it does provide a clear picture of how much this approach would cost per year.

With a sand unit price of 50 \$/m^3, and applying the same construction costs shown in Table 9, a cost of 8,102,701 \$/year is obtained. This means that in order to update the design to keep up with the expected sea level rise, a total cost of **810,270,100 $** over 100 years is estimated. When comparing this quantity to the maintenance costs due to lesser storms obtained in Section 5.3, it is **32.6 %** of this later amount. To conclude, roughly one third of the maintenance cost would need to be added to the total budget in order to prepare the design for future sea level rise.

5.5. Costs Reflection

Throughout this chapter the costs associated to the dune construction, post storm recovery, maintenance and sea level raise were estimated. Their values, along with the construction costs of the dike from *E.C. van Berchum et al., 2016* are summarized in Table 10.

Alternative	Type	Cost [billion $]
Dune design	Construction	3.061
E.C. van Berchum et al., 2016 dike	Construction	2.247
Dune design	Post storm recovery	0.296
Dune design	Maintenance due to lesser storms	2.485
Dune design	Sea level rise	0.810

Table 10. Costs summary.

Even though it may look like the dike solution proposed at *E.C. van Berchum et al., 2016* is more beneficial in economic terms, there are some aspects to take into account before reaching a final conclusion. In *E.C. van Berchum et al., 2016* only the construction costs are estimated, which entails a 26.6 % price reduction if compared to the ones obtained for the dune alternative. A saving of around a quarter of the construction budget in a project of such magnitude is not a negligible amount. Furthermore, the required partial reconstruction after a design hurricane impact slightly increases this price difference since, a traditional dike design is expected to suffer less costly damages from the design hurricane impact, although its quantification has not been obtained in the current design stage. However, an initial comparison between side, indirect and maintenance costs would help to give an insight into the total cost of the project over time.

In both *E.C. van Berchum et al., 2016* and this thesis the costs of property buyouts and connection to higher grounds are not estimated since they would require a separate in depth study, reason why they are suggested as potential follow-up topics in Chapter 7 to keep on with the Coastal Spine land barrier alternatives line of design.

Regarding long term maintenance, the dune design presented is mainly damaged by those storms intense enough to reach the dune foot and eroding its front as well as the beach in front of it. This maintenance would suppose, over 100 years, an annual expense of less than 1 % of the total construction costs as seen in Table 10. The simplicity of the dune makes it easy to predict which ones are the factors that may pose a threat to its structure, being the main one the mentioned storms. Maintenance of the dike proposed at *E.C. van Berchum et al., 2016* is not estimated in the report thus not taking into account in its cost estimation. The maintenance of the dike is not simple to estimate in advance and its design does not account with the flexibility of a dune which allows for the adoption of active measures in a short term time frame. Ageing of a dike carries problems that can be problematic to solve due to its hard structure component.

A potential risk related to the passing of time is differential settlement. Galveston Island is experiencing a mean of 2.5 cm/year land subsidence (Galloway & Coplin, 2014) which, added to the fact that the proposed dike is running along the coastal highway path, exposing it to variable traffic loads, could reduce the safety level provided by the structure. Not only a lowering of the crest level would pose a problem, an uneven variation of the crest may open cracks in the dike that in case of hurricane would expose the dike core. Furthermore, the cost of rebuilding the road after the dike construction is not taken into account neither. The presence of the road would also make maintenance works harder to perform as well as causing nuisance to the local residents due to the need of (partially) closing it. A dune design would not have this problem due to its purely sandy composition that can adapt to uneven land subsidence and by having a clear top part, not adding extra loads.

The other long term factor to be considered is the fact of keeping up with sea level rise. In *E.C. van Berchum et al., 2016* the sea level rise is mentioned but not approached. On the other hand, as obtained in Section 5.4, in order to raise the dune the expected 1 metre of relative water level increase, a budget of $0.81 billion is needed (26.4 % of the construction cost). Furthermore, the flexibility of the dune sandy solution makes it possible to distribute this cost over the years, adapting to the measured and short term expected sea level rise since long term predictions hold a lot of uncertainty. In this way, adjustments to the plan could be done over the years. Opposed to this concept, the Coastal Spine inland dike is a rigid structure needed of hard work in order to be adapted for future situations if it is not planned since the beginning, which would suppose an increased initial cost.

All of these maintenance contrast in cost could be a major difference in both initial and prolonged costs over time and further study of the Coastal Spine land barrier dike is needed before reaching conclusive arguments to support its implementation or the dune alternative one. However, a pattern in increased flexibility and more straightforward maintenance cost estimation and prediction of the dune over the dike design is repeated over the post storm recovery, maintenance and sea level rise comparison analysis.

6. Discussion

6.1. Coastal Spine Criteria Analogy

In the previous section costs of the dune alternative design as land barrier were estimated. Although the economic aspect of the project is a main factor that determines its feasibility, there are other considerations to be addressed and analysed, such as aspects involving integration and impact of the plan, that are also needed to be taken into account in order to judge based on a global perspective of the intervention instead of just a pragmatic approach.

Before addressing the aforementioned characteristics worth comparing between the Coastal Spine land barrier and the proposed natural design in this thesis, there is another feature that needs to be discussed before proceeding: dune placement within the beach cross section. It has been determined that the logical place of a dune is its natural place, the beach. However, there are two possible choices with their associated consequences. Figure 60 shows both dune location possible choices within the design cross section.

The first option is to allocate the dune in the backshore, where the current natural dune is located. This one would be the place where it fits more into the current landscape and coast configuration while being far from the erosion are induced by regular wave action. Furthermore, the use of the already existing lower natural dune is used as the base for the designed one to reduce the sand volume needed for construction and reducing its actual footprint on the beach. However, not every stretch along Galveston Bay coast is wide enough to allocate the dune without using part of private beachfront properties, being uncertain the cooperation of local owners.

The other possible solution is placing the dune closer to the coastline to avoid using private properties. This solution requires the use of more sediment (Figure 60) than estimated and would change the morphodynamic processes at the coast, potentially increasing the erosion of the coast making extra nourishment mandatory maintenance.

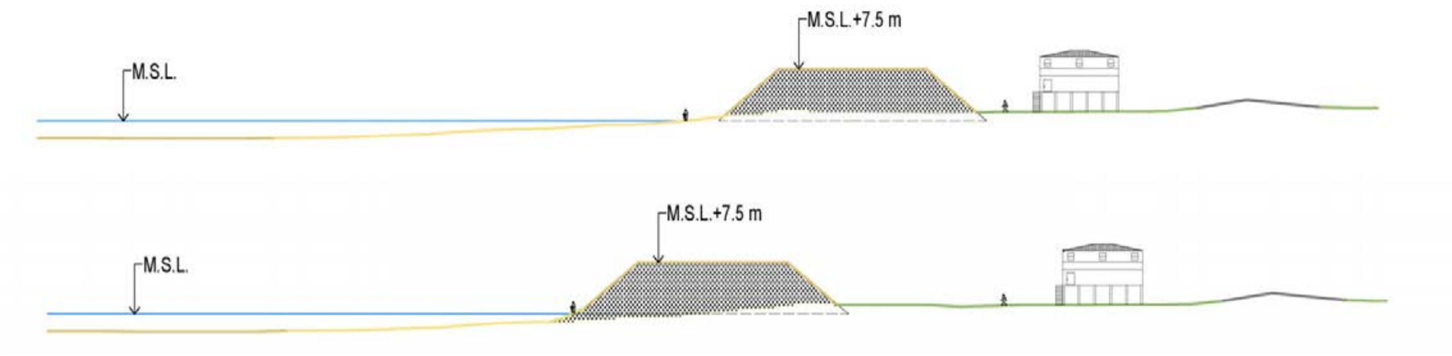

Figure 60. Design dune at the backshore (top) and foreshore (bottom).

Although a backshore dune is preferred when possible, the comparison of the different features regarding the scope of the project as well as its impact within the area will be done between the backshore dune, foreshore dune and Coastal Spine dike from *E.C. van Berchum et al., 2016*. The parameters and the score each of the land barrier alternatives scored in them can be seen in Table 11 and their explanations are given below. The score system is based on its performance or effect in the correspondent criteria, with very positive (++), positive (+), neutral (0), negative (-) and very negative (- -).

Criteria	Foreshore dune	Backshore dune	Coastal Spine dike
Vulnerability	+ +	+ +	+
Construction	-	+	-
Maintenance	- -	+	-
Flexibility	+	+	- -
Morphological impact	- -	-	0
Ecology	0	+ +	0
Landscape	+	+	-

Table 11. Land barrier alternatives scores.

Vulnerability represents the safety the structure provides against a flooding event. Obviously, since all of them are designed with the same design requirement of 1/100 conditions, this same required level of safety is achieved in all cases. The flood mitigation inside the bay is the same in all three cases and it makes a huge difference when compared with the current situation in terms of flood safety. However, in a local scale there is a significant difference, related to its placement. While the two options presented in this thesis are in the beach side, the Coastal Spine dike is placed using the same space as the coastal highways, leaving a significant portion of the coast unprotected (Figure 61). This fact is more relevant in Bolivar Peninsula. Nevertheless, the protection granted by any alternative entails a great improvement in the global flood safety of Galveston Bay compared to the current scenario.

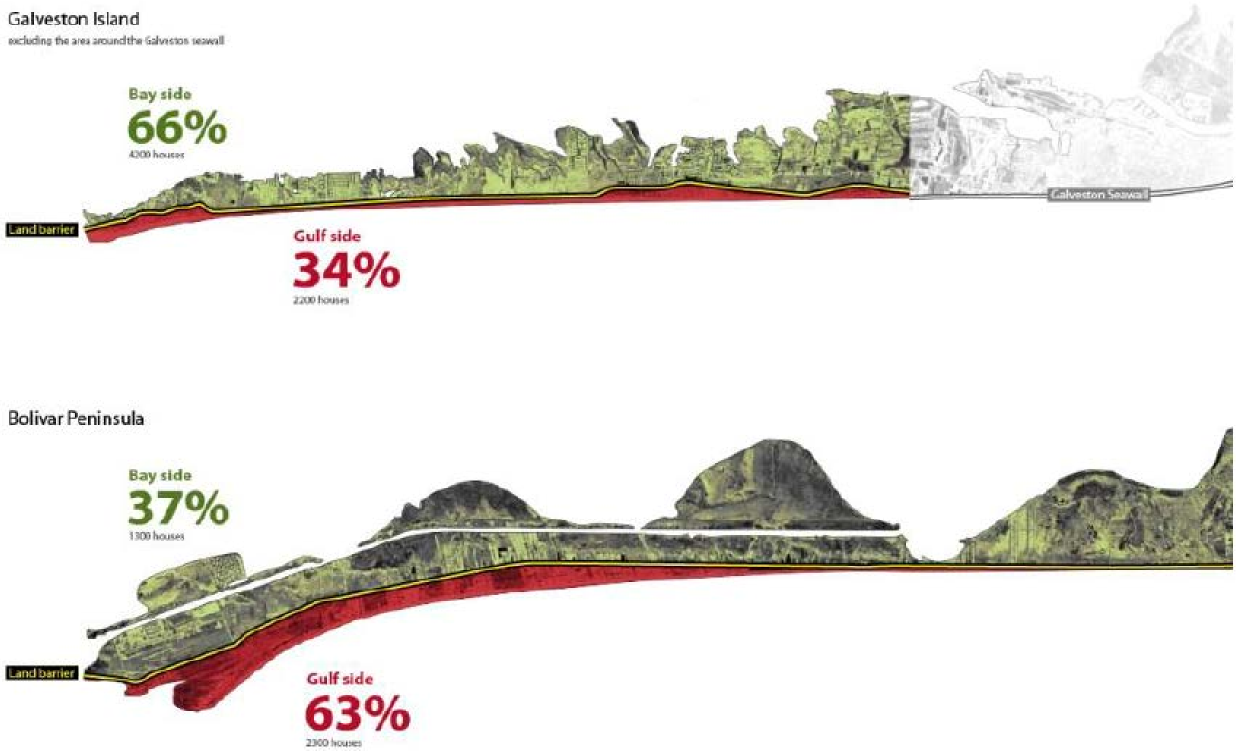

Figure 61. Relative amount of properties protected (green) and unprotected (red) in Galveston Bay coast by the Coastal Spine land barrier. Source: (van Berchum et al., 2016)

The construction criterion is the facility the process of erecting the structure. The backshore dune should not pose any major problem due to its relatively easy access and clear area without elements that may disturb the process. The foreshore dune is more complicated since water operations should be involved due to part of it being located in the current sea side of the beach. The problems with the dike construction under the road arises from the hindrance caused from it to the neighbouring properties, as well as the need to remove the current road and placing a new one on top adding a new layer of complexity to the overall building process.

The maintenance of the three land barrier alternatives are different and caused by different factors. Both dune alternatives require beach maintenance to not let beach retreat reach the dune and affect its safety, which is paramount in the whole area also due to its importance in local life quality. A dune located using the current shoreline is expected to affect the morphology, thus requiring more frequent nourishment. Maintenance of the inland dike would be needed less frequent due to less exposure to sources of erosion in regular conditions, but traffic is running on top of it, which could make the problem of subsidence and, even worse, differential settlement. Fixing these issues would mean the disturbance of traffic in the main coastal roads of the area.

An interesting feature that has not been developed in *E.C. van Berchum et al., 2016* is the need of system improvement due to expected relative sea level rise (RSLR). Over the next 100 years, a RSLR between 68 cm and 1 m is expected in the area, which calls for planning actions against it. A strong point of the soft solution proposed in this thesis is the possibility of including SLR during its regular maintenance. 1 cm of raise in the dune each year would be the preliminary needed dike height increase needed. Every time there is a programmed maintenance this raise could be included, keeping the dike updated for this boundary condition change over time.

Flexibility is the possibility of the design to adapt to future designs. This feature has been discussed in previous chapters and considered one of the strong points of this alternative. If at some point in the feature it is decided to change the strategy, the dune sand can be used in another structure or for nourishment for example. A dike is permanent and remains in place as a fixed hard structure unless more specific and destructive actions are taken. This causes the need of a robust and safer design of dikes since changing or removing it is a more difficult task.

The backshore dune is planned to be located out of sea action, and while it may slightly affect the shoreface profile, it should not be of relevant magnitude. The Coastal Spine dike is located inland. If the dune is placed in the shoreline, it drastically affects the morphodynamic of the longshore sediment transport. More in depth research should be done before committing to designs including affecting the shoreline position.

The backshore design is the one that holds the potential of including the natural approach in the adequate stretches that will be explained in the next section. This intertidal area between two dunes would enrich the natural value and enhance the current ecosystem. The inland dike would, in principle not affect too much the environmental features of the area, although a barrier effect disconnecting coastal and inland systems is still expected. In terms of landscape fitting, a soft solution like a dune would emphasize the most important feature of Galveston Bay, its beach. It would create a landmark that people would recognize as a local signature, an example of sand solutions to prevent flooding while adding extra recreational and ecology values. On the other hand the height of the road would appear as the highlight of an uninteresting feature like a highway.

6.2. Two Design Alternative

In Section 2.3 it has been explained how the Galveston Bay barrier islands are composed of parts with different features. Its coast, with over 80 kilometres of shoreline is heterogeneous regarding spatial features. With the explained spatial factors in mind, this section introduces a two design soft solution alternative as an expansion to the dune concept developed throughout the thesis. The intention of this section is to introduce how the dune approach can be the adapted to the characteristics of the area while being the starting point leading to a more nature based solution aimed to, as well as providing coastal flood safety, enhance the ecosystem values of the area.

A major characteristic that can serve as a basis to categorize different coast stretches into two groups is their degree of urbanization. Urban and rural areas are found in different parts of the coast and a single flood prevention solution that does not attend to these features is bound to not succeed in its system integration. This idea is the motivation behind the two design alternative based on dunes: one solution that focuses on the ecosystem and environmental strengths of the coast while the other is more adapted to be integrated into urban landscape, being more practical and simple. Together, these two concept designs form the dune land barrier alternative system that is homogeneous along the coast while also being adapted to the local heterogeneities.

In order to proceed with the explained concept idea of the two designs, two coastal categories has been distinguished: urban and natural. The former one is the one in front of those areas which have a reasonable amount of properties behind it. The second or natural one accounts for the rest of the coastline, in which their natural value is the priority and asset to be enhanced. A schematization of the first category selection for both islands is sketched in Figure 62 and Figure 63.

Figure 62. Galveston Island coast spatial characterization.

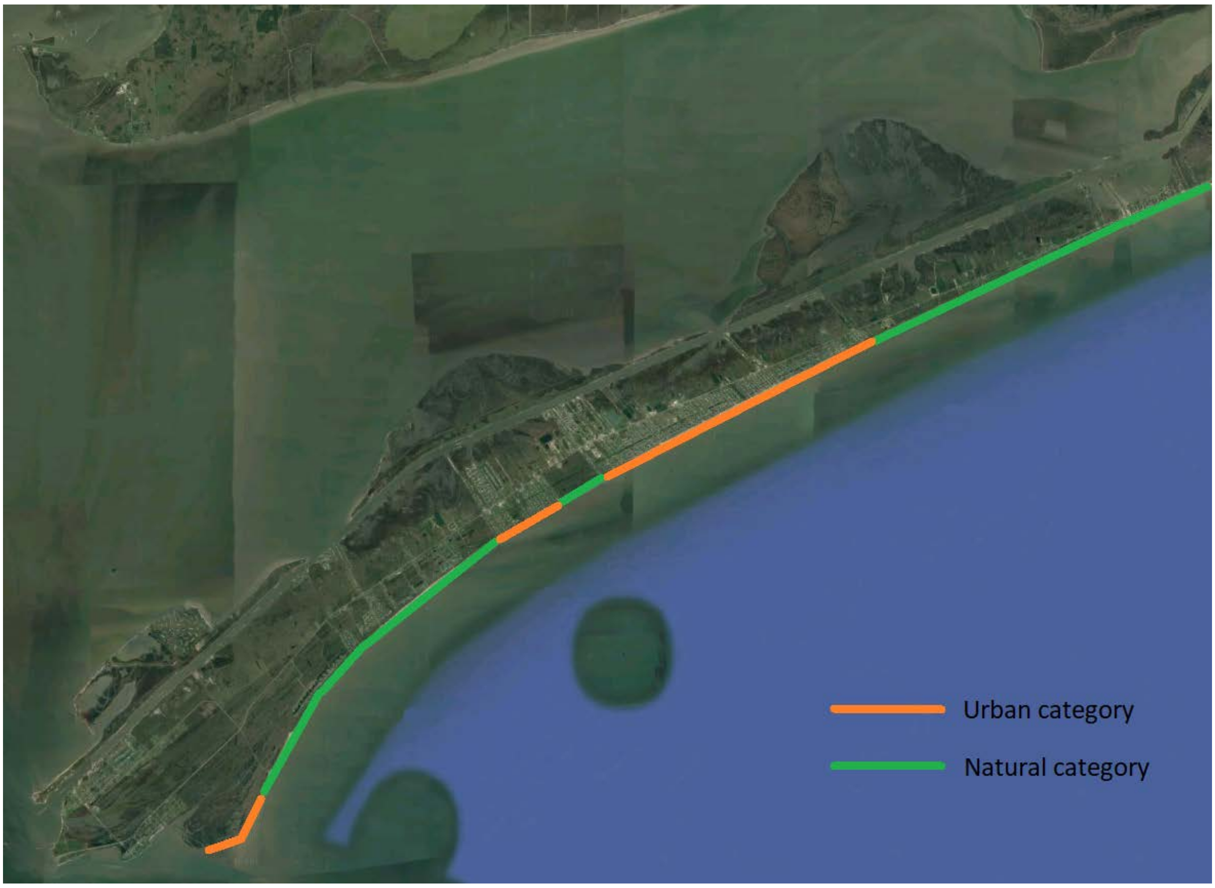

Figure 63. Bolivar Peninsula coast spatial characterization.

The dune to be placed in the urban category areas is the one developed throughout this thesis, while a similar concept of the nature enhancing dune is found in one of the barrier alternatives in previous Coastal Spine reports (van Berchum et al., 2015), showed in Figure 64. However, some differences apply from these concepts to the proposed in this report. The main dike and detached breakwater in Figure 64 in the idea proposed in this thesis would be changed for a main dune (similarly characterized as the design dune) and a detached dune. The area between the two dunes would provide an intertidal zone where vegetation and species could develop in this rich and dynamic environment.

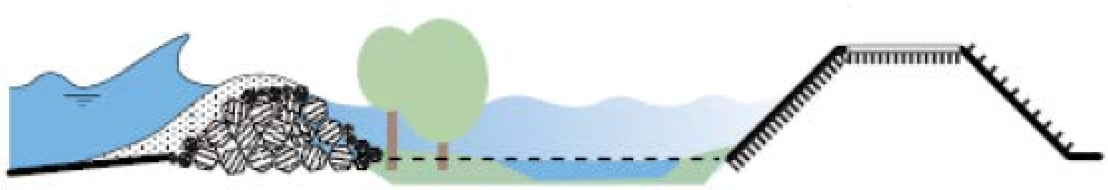

Figure 64. Concept design of nature enhancing flood defence. Source: (van Berchum et al., 2015)

This nature based design would have a large footprint (more than 100 metres) in order to be effective, thus the shoreline would be artificially changed to a more offshore position and a change of erosion rates would be expected. More study into this morphodynamic changes is required in order to quantify the impact of such intervention. A visual representation of this conceptual design alternative idea, comprising a representative section where the nature enhancing dune would be implemented, giving extra focus on the local ecosystem improvement, is depicted in Figure 65.

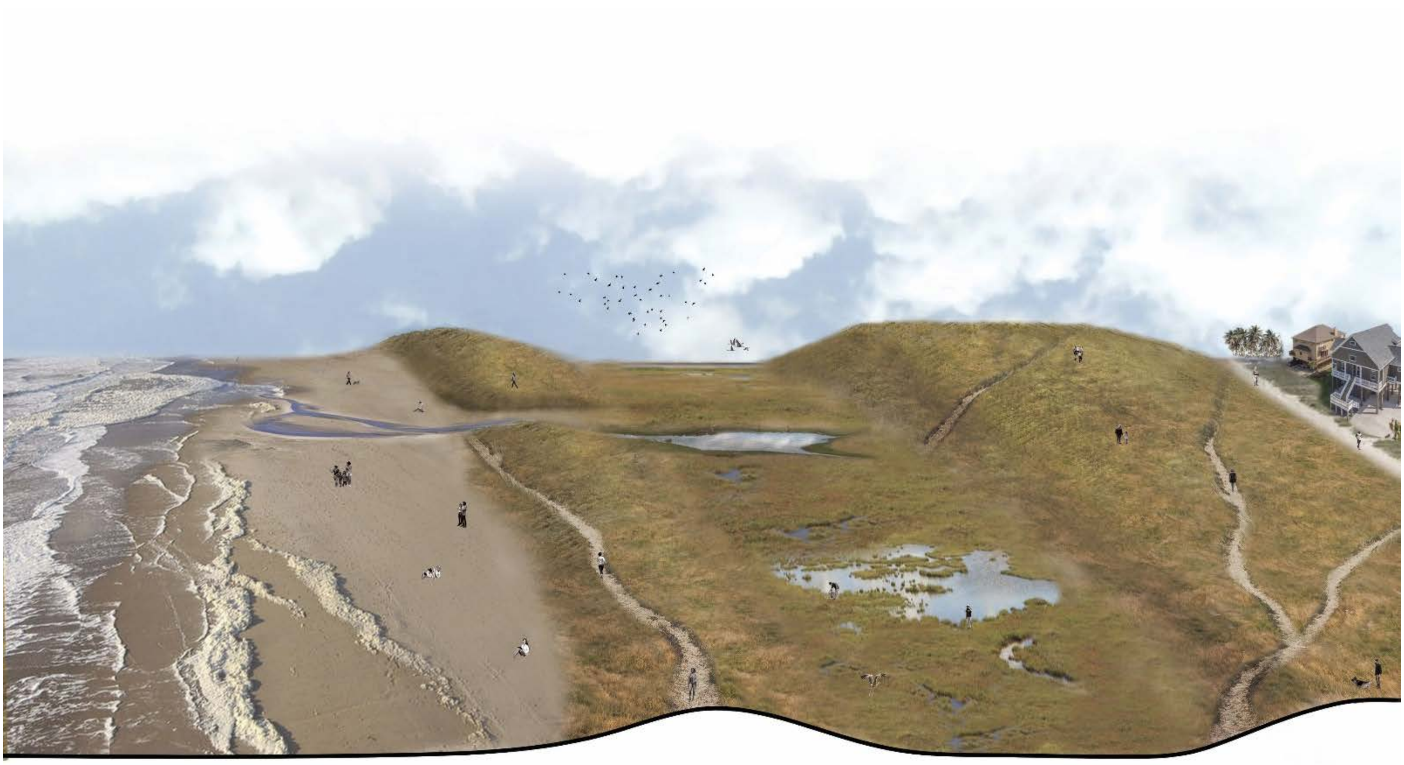

Figure 65. Illustration of the nature enhancing flood protection dune concept design.

The design of this nature focused dune must be dimensioned following the same design boundary conditions as the design dune, being able of withstanding the surge and wave forcing caused by the design 1/100 yr^{-1} hurricane. The dune design criteria along the coast must be uniform since failure (breach, overflow…) of one part would potentially lead to a cascade effect where adjacent sections are more bound to fail as well. The visual conceptual profile development through the design storm of this nature focused defence is depicted in Figure 66.

Figure 66. Illustration of the nature enhancing flood protection dune at the beginning (left) and during the storm (right).

A beneficial side effect that could play an important role in flood risk reduction is the resilience effect the detached dune has in the Figure 66 design after this detached dune has failed during a hurricane event with enough intensity. In that case early wave breaking due to reduced water depth in that zone would take place, thus reducing the erosion in the main line of defence dune since waves would break far from the main dune, having already dissipated most of their energy before reaching the front of the main dune.

7. Conclusions and Recommendations

7.1. Conclusions

The objective of this thesis is to design a dune system that can serve as a land barrier in the Coastal Spine plan, evaluate its performance under design hurricane conditions and reach a conclusion about its feasibility compared to the dike proposed in *E.C. van Berchum et al., 2016*.

A dune based system is presented as an alternative to the Coastal Spine land barrier dike that can work as a coastal flood defence structure in case of hurricane. Advantages of the dune system over a standard dike design are design flexibility, landscape integration, ecology and natural values among others. A dune system running parallel to Galveston Island and Bolivar Peninsula would enhance the relevance coast has in Galveston Bay inhabitants and economy while protecting the area from the terrible consequences in case of a high category hurricane such as the ones that took place in 2008 with Hurricane Ike.

A 1/100 design safety level is chosen due to the potential design flexibility compared to the storm surge barrier rigid design. Thus an initial design storm surge of 5.2 m and offshore significant wave height of 5 m are used. The main initial parameters defining the dune design are its basic dimensions: height, width and slope. Crest height is obtained by designing for overtopping criterion, obtaining an allowable overtopping volume of 44.3 l/m/s during hurricane peak conditions with a crest level of MSL +7.5 m. Excessive overwash is not allowed to prevent a rapid dune wash away. Width is obtained by calculating expected dune recession during the storm event and accounting for safety margins, a value of 100 m width at MSL is estimated, with 55 m of crest width. A slope with 1:3 ratio is selected as the dune natural angle.

After the initial dune parameter characterization is done, XBeach numerical modelling is used to evaluate the design and assess its performance during design conditions, determining whether the design is valid, needs to be improved or optimized. Storm surge and wave condition time series are estimated from recorded Ike measurements. The final model shows interesting results during the simulation, where steady erosion of the dune at water level takes place during the hurricane forerunner phase, while at the peak of the storm abrupt loss of sediment is observed across the whole dune front. This correlation leads to the early conclusion that dune width is needed to withstand the duration of the storm while dune height is responsible of withstanding the peak surge fast load. The total amount of dune erosion caused by the impact of the design 1/00 hurricane is 187.5 m^3/m. In the model vegetation is implemented and its effect is noticeable at the dune foot, where prevents part of its erosion. Its inclusion in the design also helps to its integration with the landscape. After the design storm, around 40 % of the dune is still erect, which is considered an adequate safety margin considering the over 80 kilometres of coast that is needed to be protected.

To assess the feasibility of this thesis design, its implementation costs are estimated and a comparison is done with the most recent and updated Coastal Spine land barrier dike design, developed in *E.C. van Berchum et al., 2016*. The main difference to be accounted for when comparing costs is that a dune is expected to be eroded during storm and its recovery is crucial for the flood safety of the area. This post storm recovery costs is added to the estimated construction ones, having a combined value of $ 3.318 billion. The Coastal Spine dike design is 67.7 % of this value. However, the long term costs including to maintenance due to lesser storms and sea level rise have not been estimated in previous Coastal Spine land barrier reports while for the design dune system the combined cost is estimated to be $ 3.295 billion. Thus, in order to maintain the dune system along its 100 year design life an approximate yearly cost of 1 % of the construction costs must be invested. It is believed that the flexibility and simplicity behind the dune system concept as opposed to a traditional dike would make the long term maintenance of the former one economically more attractive, fact that would balance the initial construction costs difference between the two approaches.

Other criteria such as vulnerability, maintenance or flexibility of the two designs are compared, dividing the dune location into two possible options: backshore and foreshore to account for possible issues regarding private land buying costs. In general terms, the backshore dune yields slightly better results overall and is concluded that a backshore located dune is preferable when possible. Nonetheless, this qualitative assessment is bound to subjective judgement based on knowledge in the case.

An interesting potential feature of the dune concept is presented: the possibility of implementing a two design alternative instead of a single standard dune design. This concept idea is proposed attending at the features of the different coast sections, applying the design that fits better in each place. The first dune design is the one that has been developed throughout the thesis, focusing mainly in the flood protection aspect. Within this approach, it can be considered as a traditional dune design, a pragmatic dike alike dune that serves as a water retaining structure in a coastal flooding scenario. The second one, more innovative and with lots of building with nature potential, includes a second dune line in the front, seawards. The area left in between is conceived as a dynamic intertidal area where the ecosystem can expand towards, enhancing the natural value of the area. In depth research, analysis and design would be needed in order to assess the viability of this approach since the coast morphodynamic would be drastically altered.

It is now possible to provide an answer to the main research question:

Is it feasible to implement a coastal flood risk reduction plan for Galveston Bay with a dune system instead of a hard structure as land barrier?

Preliminary results indicate that the possibility of a dune system functioning as flood defence structure can work adequately if designed accordingly to its distinctive features. In this particular case, the dimensions of the dune are within reasonable values with respect to previous Coastal Spine designs, and modelling results show satisfactory performance against design scenarios, providing reliable safety margins. A traditional dike, world extended structure, could fulfil the same flood risk reduction functions while its construction price is two thirds of the dune design construction and recovery costs combined. However, having analysed and quantified the dune system long term costs, compared to the possible dike maintenance costs, they are likely to compensate for the higher initial construction costs of the former design implementation. At this stage it is hard to assess whether the added values provided by a dune system (nature, maintenance, landscape, acceptance…) compared to a dike are enough to justify the cost difference. In order to provide conclusive verdict, quantification and detailed analysis of this dune approach peculiarities, as well as estimation of maintenance of the dike alternative, is required to be performed before committing to this innovative design that certainly holds potential.

7.2. Recommendations

This thesis opens different areas of research within the treated topic:

- The 1/100 safety level has been the followed standard in the U.S. and in this thesis as well, but hurricanes with **higher return periods** could be taken as design boundary conditions. In The Netherlands the safety standard for flood risk design is in general some order of magnitude higher, taking this principle to Galveston Bay case could lead to a more optimized strategy. Modelling hurricanes with different boundary conditions and trajectories in a 2D scenario would give valuable information about how the dune system reacts to a variety of scenarios and the possibility of optimize it for different scenarios.

- The location of the dune within the cross section still poses a problem in every design done so far due to the uncertainty on property owners cooperation. Placing the **dune at shoreline** reach would avoid this issue since coastal land is of public domain. A change in the morphology is expected but it is uncertain up to what extent. Research on the change of the sediment transport could result in a favourable situation where it is more convenient shifting the coast than using private property lands. Optimising the placement of the dune within the cross section could suppose a great economic impact in the project budget by finding a balance between private buyout costs and maintenance costs due to the exposure of the dune to areas experiencing more erosion. It would also be interesting the rate at which, after a hurricane event, the nearshore accreted area returns to the beach naturally by morphodynamic processes. If it is fast enough, recovery dredging actions would be optimised by not needed an urgent intervention.

- Along with the previous topic, the development of the **nature enhancing design** exposed in Chapter 6 would give a strong argument supporting the feasibility of this design. Designing the lengths at which it could be applied, the intertidal area, the dimensions of the detached dune or the openings needed to keep the flow in constant movement in an out of the area are parameters that need to be defined in order to proceed with further development. Since this design needs a considerable amount of space, it would extend into the sea, changing the morphology of the coast, thus affecting its morphodynamic behaviour. Knowledge on the longshore sediment transport and how is affected is extremely important to understand and fight the beach retreat situation at Galveston Bay coast. Furthermore, more sand supply would be needed since the nature focus design requires more sand volume, thus exploring and optimising the possible sand supply options and reducing its cost would greatly benefit its feasibility.

- The weakest points of a structure which function is to retain water is its **connections** to other systems. In this case there are two connections to make: with the storm surge barriers and with the west and east ends. In the case of the design ends, the connection would need to be done to higher grounds to achieve total closure of the bay in case of hurricane. Technical design of these features is needed since early failure at those points would lead to a cascading effect where breaching would also take place in adjacent sections. Another similar aspect of the project would be the integration of the dune design in the Galveston Seawall stretch. The concept of ‘dike in dune’ where, in this case the seawall, is covered by a layer of sand seems a reasonable starting point for a design that would not break with the overall philosophy of the system.

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Appendix A - Overtopping

A dune, when it is seen as a mean of coastal defence system such in this case, can be approached as a dike, apart from the erosion it experiences during its lifetime. In the current situation, the fact that the structure is erodible does not affect the way overtopping takes place. The processes of shoaling, wave breaking and run-up still take place in the same way it would in a dike.

The process followed to obtain the overtopping discharge is the one in the current design guideline manual on wave overtopping: *Manual on wave overtopping of sea defences and related structures* (Van der Meer et al., 2016).

The formula used to estimate the mean overtopping discharge is the following one:

$$\frac{q}{\sqrt{g \cdot H_{m0}^{3}}} = \frac{0.023}{\sqrt{\tan\alpha}} \cdot \gamma_b \cdot \xi_{m-1,0} \cdot \exp\left\{-\left(2.7 \cdot \frac{R_c}{\gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \gamma_v \cdot \xi_{m-1,0} \cdot H_{m0}}\right)^{1.3}\right\}$$

Equation 2. Mean overtopping discharge.

With a maximum of:

$$\frac{q}{\sqrt{g \cdot H_{m0}^{3}}} = 0.09 \exp\left\{-\left(1.5 \cdot \frac{R_c}{H_{m0} \cdot \gamma_f \cdot \gamma_\beta}\right)^{1.3}\right\}$$

Equation 3. Maximum overtopping discharge.

The Iribarren number $\xi_{m-1,0}$ defines the breaking wave type and is defined as:

$$\xi_{m-1,0} = \frac{\tan\alpha}{\sqrt{H_{m0} / L_0}}$$

Equation 4. Iribarren number formula.

The deep water wave length L_0 is defined in Equation 5. In this case the design peak period will be used as reference value to represent the behavior of the storm waves.

$$L_0 = \frac{gT^2}{2\pi}$$

Equation 5. Deep water wave length formula.

The crest freeboard R_c is defined as the crest height of the dune relative to the water level. In this case is the difference between the crest level of the dune and the design surge level.

There are four safety factors which may influence overtopping reducing the discharge: γ_b, γ_f, γ_β and γ_v each of them associated to the influence of a berm, friction, oblique wave incidence and vertical wall respectively. The design is made to resist a critical scenario where the wave attack is perpendicular to the dune, and the placement of a hard structure such a vertical wall on top of the dune works against the philosophy of the design of not using elements involving hard structure components and overall is a not recommended practice.

However, when it comes to berm and vegetation influence there is potential design improvement that can be achieved. Although in this initial overtopping assessment these factors remain unaffected, in Chapter 4 they will be the basis of potential design optimization. Furthermore, not accounting for them leaves a margin of safety in the results assessments at this stage.

Using the 1/100 design boundary condition, the aforementioned equations and an initial 7.5 m design crest level, it is possible to calculate the overtopping discharge. The initial data, results of the procedure calculations and result is given in Table 12.

Parameter	Value	Units
Storm surge	5.2	MSL + m
Crest level	7.5	MSL + m
Freeboard, R_c	2.3	m
Significant wave height, H_{m0}	5	m
Wave peak period, T_p	14	s
Deep water wave length, L_0	306.017	m
Dune slope, α	18.43	o
Iribarren number, $\xi_{m-1,0}$	2.37	-
γ_b / γ_f / γ_β / γ_v	1	-
Overtopping discharge, q	0.0443	$m^3/m/s$
Maximum overtopping discharge, q_{max}	0.5357	$m^3/m/s$

Table 12. Overtopping assessment results.

As it can be seen, the overtopping discharge for a crest level at MSL+7.5 m is 0.0443 $m^3/m/s$ (44.3 l/m/s). The calculations are repeated for crest levels of MSL+5.2 m and MSL+10 m, with overtopping discharge values of 648.62 l/m/s and 0.601 l/m/s respectively. The assessment of the crest height design within its context using the obtained overtopping results, attending to the case requirements and features, is done in Section 3.2.

Appendix B – Storm Dune Recession

As explained in Section 3.3, the factor governing the initial design dune width dimension is the retreat or erosion that takes place during the design hurricane event. In order to estimate the erosion taking place during the design event, the simplified dune erosion rule (DUNERULE model) is used (van Rijn, 2013), which is obtain through a sensitivity analysis and of different runs from the process based CROSMOR model, parametrizing mathematical model results of different scenarios.

The erosion rule reads as follows:

$$A_{d,t=5} = A_{d,ref}\ (d_{50,ref}/d_{50})^{\alpha 1}\ (S/S_{ref})^{\alpha 2}\ (H_{s,o}/H_{s,o,ref})^{\alpha 3}\ (T_p/T_{p,ref})^{\alpha 4}\ (\tan\beta/\tan\beta_{ref})^{\alpha 5}\ (1+\theta_o/100)^{\alpha 6}$$

Equation 6. Simplified dune erosion rule.

The different parameters of the formula, including the reference parameter values, are:

$A_{d,t=5}$ = dune erosion area above storm surge level after 5 hours (m^3/m),
$A_{d,ref}$ = dune erosion area above S storm surge level after 5 hours in Reference Case= 170 (m^3/m),
S = storm surge level above mean sea level (m),
S_{ref} = storm surge level above mean sea level in Reference Case= 5 (m),
$H_{s,o}$ = offshore significant wave height (m),
$H_{s,o,ref}$ = offshore significant wave height in Reference Case= 7.6 (m),
T_p = peak wave period (s),
$T_{p,ref}$ = peak wave period (s) in Reference Case= 12 (s),
d_{50} = median bed material diameter (m),
$d_{50,ref}$ = median bed material diameter in Reference Case= 0.000225 (m),
$\tan\beta$ = coastal slope gradient defined as the slope between the -3 m depth contour (below mean sea level) and the dune toe (+3 m),
$\tan\beta_{ref}$= coastal slope gradient defined as the slope between the -3 m depth contour and the dune toe (+3 m) for the Reference Case= 0.0222 (1 to 45),
θ_o = offshore wave incidence angle to coast normal (degrees),
α_1 = exponent=1.3,
α_2 = exponent=1.3 for $S<S_{ref}$ and α_2=0.5 for $S>S_{ref}$,
α_3 =α_4=α_6=0.5 (exponents),
α_5 = exponent=0.3.

With this formulation the erosion area after 5 hours of storm in the design cross section is obtained. To estimate the correspondent dune recession after a different period of time two transformations have to be done:

$$A_{d,t}=A_{d,t=5}\ (t/t_{ref})^{\alpha 6}$$

Equation 7. Erosion area equivalent for different time frames formula.

$$R_d=A_d/(h_d\text{-}S)$$

Equation 8. Horizontal dune recession formula.

With:

R_d = average horizontal dune recession (m),
h_d = height of dune crest above mean sea level (m)
t = time in hours (t_{ref}= 5 hours),
α_6 = exponent= 0.5 for $t<t_{ref}$ and 0.2 for $t>t_{ref}$.

The time chosen for this step is 2 hours. As it can be seen in section 4.3.1, the maximum or peak storm surge of hurricanes with this order of magnitude is not maintained over time and is barely prolonged over a relatively short time span. 2 hours of constant design significant wave height action against the dune is a conservative initial value which will likely will suffice for an initial estimation and will provide a reliable safety margin; in later stages of the design optimization of the dune width can be performed.

Table 13 summarizes the input parameters and results obtained from the application of the dune erosion rule.

Parameter	Value	Units
Storm surge level above mean sea level	5.2	m
Offshore significant wave height	5	m
Peak wave period	14	s
Median bed material diameter	0.0001323	m
Coastal slope gradient	0.0186	s
Offshore wave incidence angle to coast normal	0	º
Dune erosion area above storm surge level after 5 hours	198.767	m^3/m
Time	2	h
Dune erosion area above storm surge level after 2 hours	125.712	m^3/m
Height of dune crest above mean sea level	7.5	m
Horizontal dune recession	54.65	m

Table 13. Dune recession results.

The assessment of the obtained dune recession value of 54.65 m and its implications is discussed in Section 3.3, where an initial design dune width is established.

Appendix C – XBeach Validation

In order to ensure the reliability of Xbeach model results, it is recommended to perform a validation process that compares model output with real data from the study case. If both datasets are comparable in terms of magnitude, trend and behavior, it can be concluded that the model captures well the physical process in that particular context.

In this case, hurricane Ike is an event that serves perfectly as a basin for model validation. Not only took place in the same area and with relative similar magnitude to the design scenario, but there in situ water level data is available via sensor sites deployed over Galveston Bay coast (East, Turco, & Mason, 2008) from the times of the event. Particularly, Station SSS-TX-GAL-010 (Figure 67) is located only two kilometres east from the dune design location (Section 4.2). Its proximity to the point of interest reinforces its suitability as a reliable validation model input.

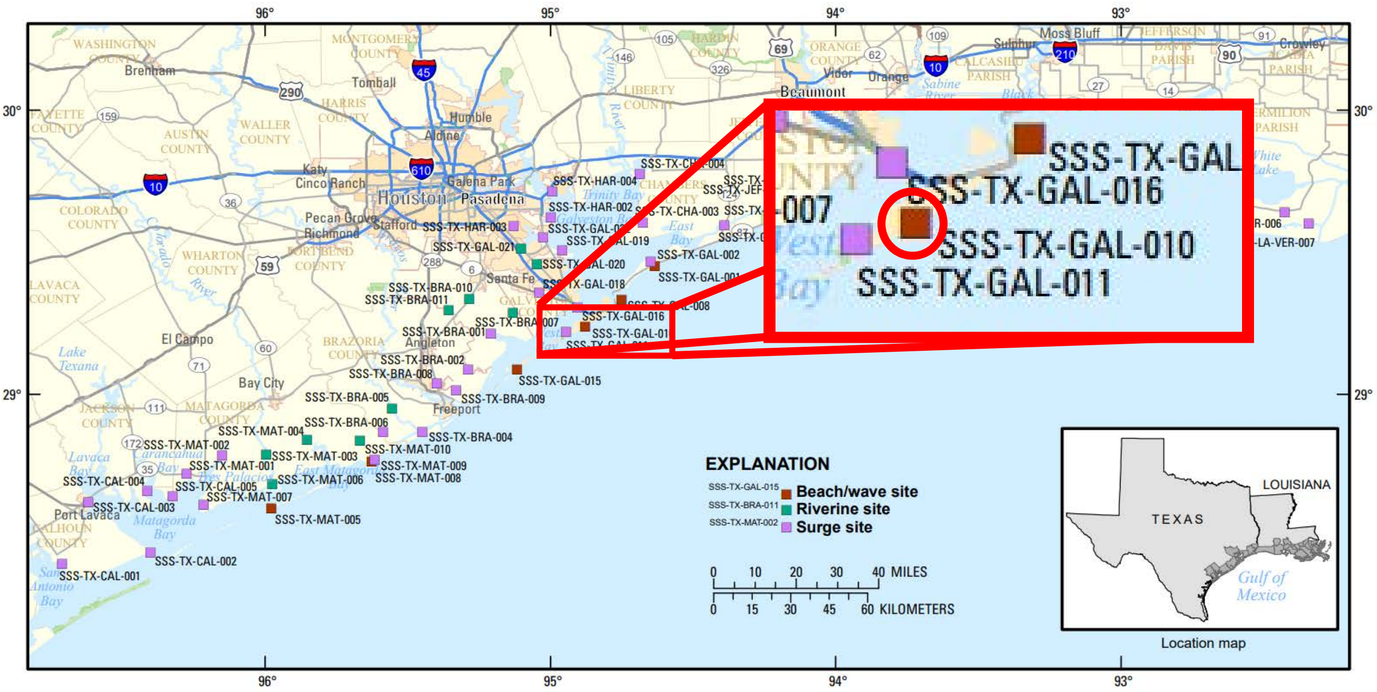

Figure 67. Station SSS-TX-GAL-010 location in Galveston Island. Source:(East et al., 2008)

As for wave data, the Ike parameters exposed in Section 4.3.2 can be used in this simulation since they are obtained offshore. Since wave variation is assumed to be similar within the same longshore transect, the same values (Figure 35 and Figure 37) recorded by Station 42035 (Figure 33) are used. The topography at the validation location before and after Hurricane Ike impact is be retrieved from (NOAA, 2008).

The validation process in this case consists of two parts, first a comparison of the hydrodynamic components and later an analysis of the land morphology. With the first part is ensured that the right loads associated with certain surge levels are taking place, while the second one assures that the morphodynamic response to the hydraulic loads of the model and measurements are comparable between each other.

The hydrodynamic component validation results obtained from the model and its comparison with the measured data at Station SSS-TX-GAL-010 location is depicted in Figure 68.

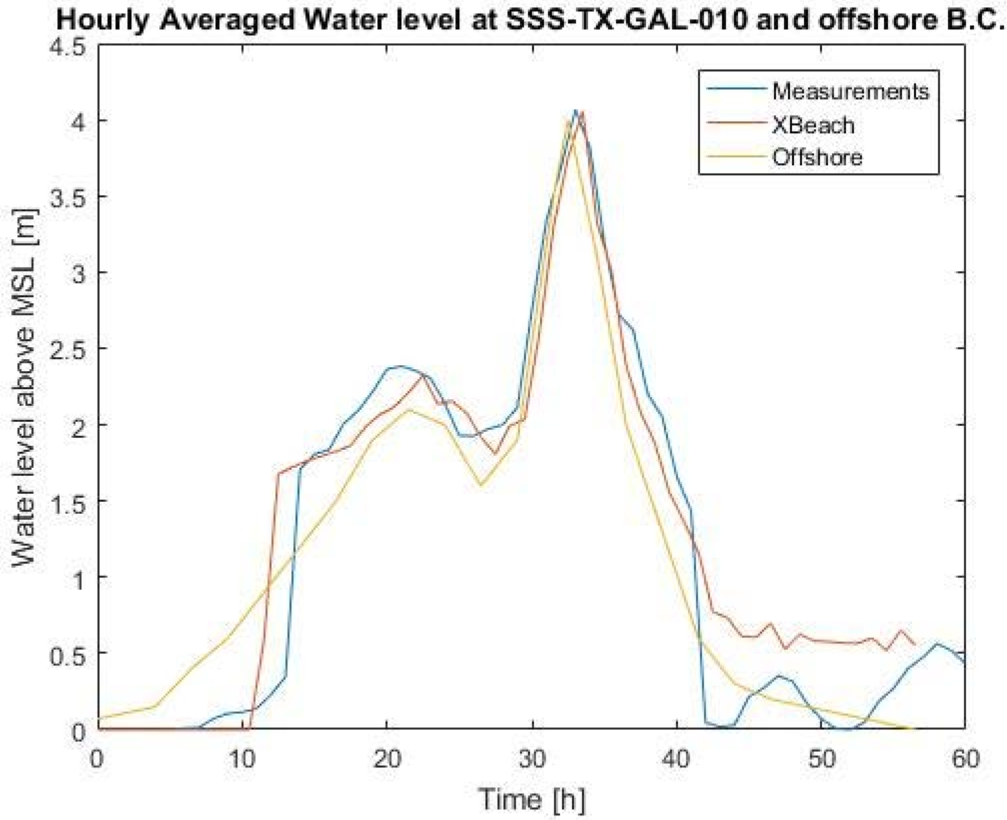

Figure 68. Water levels at Station SSS-TX-GAL-010 from in situ measurements (blue), Xbeach model results (red) and offshore (yellow).

Although during peak level all of the data have extremely similar values, during the other stages of the hurricane the surge behavior at the station deviates from the offshore one. The forerunner at the shoreline is higher than offshore, due to shallow water transformations. This forerunner water level increase is well captured by the model, relatively resembling the measured values. This level increase seen is does not take place at peak surge because this raise in the water level makes the water depth increase, so the conditions are not those of shallow water anymore.

It is expected that the water level increase during the hurricane early stages is progressive offshore while being more sudden in the inland coast, the land features at the beginning of the coast can retain up to water level, and from there the inundation takes place rapidly. Lastly, the water level at the late stages of the hurricane is not captured well by XBeach, due to limitations in the topography inland boundary in the model. However, this part is not of major relevance in our case since the dune that is deigned will not allow water to go through the inland parts. Furthermore, the differences are in the order of magnitude of half metre and in the value range of 0 and 1 metres, these magnitudes have really low impact on the dune compared to the ones experienced during peak and forerunner times.

Once the hydrodynamics have been validated, it is possible to analyse whether the model morphology reacts to the forcing in the same way as the measurements. This behavior is especially relevant in the dune area, since it is the focus of the project. The initial and final model topography together with the measured topography before and after Ike are displayed in Figure 69.

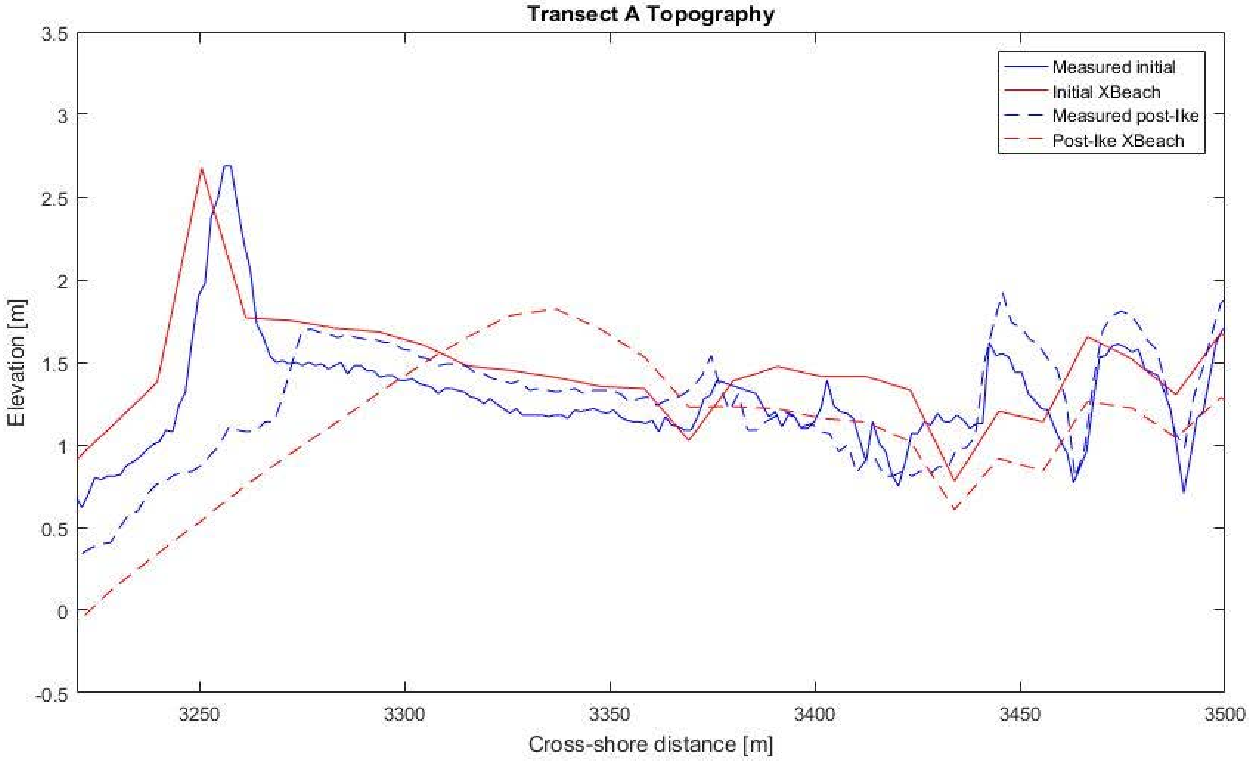

Figure 69. Topography at Station SSS-TX-GAL-010 transect from in situ measurements before (solid blue) and after (dashed blue) Ike and Xbeach model results before (solid red) and after (dashed red) Ike.

Due to limitations in the datasets, two pairs of them are to be compared when assessing their validity. In the measured topography it is seen how the Ike impact washed away the natural dune completely and just left a minor step in the bed level resembling a secondary dune. However, in the model output it can be seen how the material eroded from the dune is deposited 100 metres inland. The difference in behaviours is due to the one year difference between the measured topography datasets, time enough for human intervention to take place and remove the sand from the hinterland places that it should not be. What it is actually comparable is that erosion takes place in a similar way during the storm and that when the natural dune cannot withstand overflow, it is completely washed away in both cases.

It is concluded that, for the current case study, with the used input parameters the Xbeach model captures well enough the storm behavior and morphodynamic changes like dune erosion associated to the hydraulic forcing.